AMERICAN CONCRETE INSTITUTE

PROCEEDINGS

OF THE

FIFTEENTH ANNUAL CONVENTION

Held at Atlantic City, N. J.,

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BY-LAWS.

Article I.

MEMBERS.

Section 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A firm or company shall be treated as a single member.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at that time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

Sec. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

Article II.

OFFICERS.

Section 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

Sec. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

Sec. 3. There shall be a Committee of five members on Nomination of Officers, elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall select by letter ballot of its members, candidates for the various offices to become vacant at the next

Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

Sec. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-Presidents and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty, of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

Sec. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

Sec. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

Sec. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President at the first regular session of the Annual Convention.

Sec. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

Sec. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

Sec. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

Sec. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

Sec. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

Sec. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

Article III.

MEETINGS.

Section 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of the Convention.

Sec. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

Sec. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

Sec. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

Article IV.

DUES.

Section 1. The fiscal year shall commence on the first of July and all dues shall be payable in advance.

Sec. 2. The annual dues of each member shall be ten dollars ($10.00).

Sec. 3. Any person elected after six months of any fiscal year shall have expired, need pay only one-half of the amount of dues for that fiscal year: but he shall not be entitled to a copy of the Proceedings of that year.

Sec. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon the payment of all indebtedness against them upon the books of the Institute.

Article V.

RECOMMENDED PRACTICE AND SPECIFICATIONS.

Section 1. Proposed Recommended Practice and Specifications to be submitted to the Institute must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter; such Recommended Practice and Specifications shall be considered adopted unless at least 10 per cent of the total membership shall vote in the negative.

Article VI.

AMENDMENT.

Section 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF THE PROCEEDINGS OF THE FIFTEENTH ANNUAL CONVENTION.

Hotel Traymore, Atlantic City, N. J.

First Session, Friday, June 27, 1919, 10.30 a. m.

The convention was called to order by W. K. Hatt, President of the American Concrete Institute.

The report of the Committee on Plain and Reinforced-Concrete Sewers was presented by Coleman Meriwether, a member of the committee. After a long discussion on the proposed standard specification for monolithic concrete sewers and reinforced-concrete pipe sewers, and recommended rules for concrete sewer design, the report with its attendant specifications was referred back to the committee for reconsideration, to be resubmitted at the next convention.

The report of the Committee on Treatment of Concrete Surfaces was read by its chairman, J. C. Pearson. The accompanying Standard Recommended Practice for Portland Cement Stucco was accepted by the convention to be printed in the Proceedings.

The following papers were read and discussed:

"Effect of Vibration, Jigging, and Pressure on Fresh Concrete," by Duff A. Abrams.

"Investigation into the Economic Possibilities of Light-Weight Aggregates in Building Construction," by A. W. Stephens.

The session adjourned to attend the afternoon session of the American Society for Testing Materials held in same hotel, 2.00 p. m.

Second Session, Friday, June 27, 1919, 8.00 p. m.

Joint meeting with the American Society for Testing Materials. President W. K. Hatt, of the American Concrete Institute, and President G. C. Clamer, American Society for Testing Materials, in the chair.

The report of the A. S. T. M. Committee C-1 on Cement was presented by its chairman, R. S. Greenman.

The report of the A. S. T. M. Committee C-9 on Concrete and Concrete Aggregates was presented by its chairman, Sanford E. Thompson.

The report of the American Concrete Institute Committee on Fireproofing was presented by its chairman, W. A. Hull.

The following papers were read and discussed:

- American Concrete Institute paper—"Fire Tests of Concrete Columns," by W. A. Hull.
- American Concrete Institute paper—"The Strainagraph and Its Application to Concrete Ships," by F. R. McMillan.
- American Society for Testing Materials paper—"Effect of Fineness of Cement," by D. A. Abrams.
- American Society for Testing Materials paper—"Cement Producing Quick-Hardening Concrete," by P. H. Bates.

THIRD SESSION, SATURDAY, JUNE 28, 10.00 A. M.

President W. K. Hatt in the chair.

The report of the Committee on Reinforced Concrete and Building Laws was read by its chairman, E. J. Moore. The report was received as a progress report and will be printed in the Proceedings.

The report of the Subcommittee on Regulations for Strength Test of Floors was presented by its chairman, W. A. Slater.

Mr. Richard L. Humphrey in the chair.

The following papers were read and discussed:

- "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation," by W. A. Slater.
- "Elasticity and Temperature Deformations in Concrete," by S. C. Hollister.

The report of the Committee on Reinforced-Concrete Highway Bridges and Culverts was presented by its chairman, A. B. Cohen. The report was accepted as a progress report and will be printed in the Proceedings.

The paper entitled, "Concrete Work on the Brooklyn Army Base," by A. C. Tozzer, was presented by E. J. Moore.

FOURTH SESSION, SATURDAY, JUNE 28, 1919, 2.00 P. M.

President W. K. Hatt in the chair.

Business Session.

The Secretary read the report of the Board of Direction.

The Treasurer read the report of the Treasurer.

The Secretary reported that the letter ballot had resulted in the election of the following officers for the ensuing year:

President: William K. Hatt.
Vice-President: Sanford E. Thompson.
Treasurer: Robert W. Lesley.
Directors: *Third District*—Ernest Ashton.
Fourth District—W. P. Anderson.
Fifth District—W. M. Kinney.

The report of the Committee on Building Blocks and Cement Products was presented by the chair. The report was received and the specification on the Manufacture of Concrete Roofing Tile submitted to letter ballot as one of the standards of the Institute.

The report of the Committee on Concrete Roads and Pavements was presented by its chairman, H. E. Breed. The report was received and ordered printed in the form of proposed amendments to the proposed specifications of last year.

At this point the proceedings were interrupted long enough to read the announcement of the signing of the treaty of peace at Versailles. The members rose and joined in singing the national anthem.

The announcement of the award of the Wason Medals was made by the chairman of the Wason Medal Committee, F. C. Wight. The awards were made for three years, as follows: 1916, to A. B. MacDaniel, for his paper, "The Effect of Temperature on Strength of Concrete;" 1917, to Lt.-Col. Chas. R. Gow, for his paper, "The History of Concrete Piles;" 1918, Prof. Duff A. Abrams of Chicago, for his paper entitled, "Effect of Time of Mixing on the Strength and Wear of Concrete." Mr. L. C. Wason, the donor of the medals, presented them to the recipients.

The following papers were read and discussed:

"The Design of Reinforced-Concrete Fuel Oil Reservoirs," by H. B. Andrews.

"Tests of Concrete Tanks for Oil Storage," by J. G. Pearson and G. H. Smith.

The report of the Committee on Concrete Sidewalks and Floors was presented by its chairman, J. E. Freeman. The report as amended was received and the proposed revisions on specifications on concrete floors and sidewalks ordered submitted to letter ballot of the Institute.

A paper entitled, "Some Remarks on Earthquake Resisting Construction in Central America," by Juan I. DeJongh, was read by title.

The report of the Committee on Nomenclature was read by the chairman, W. A. Slater. It was received as a progress report and will be printed in the Proceedings.

SIXTH SESSION, SATURDAY, JUNE 28, 1919, 8.00 P. M.

Mr. Sanford E. Thompson in the chair.

The following papers were read and discussed:

"Concrete Railroad Track," by A. C. Irwin. Read by J. E. Freeman.

"The Development of Concrete Ships," by J. E. Freeman.

The final report of the Committee on Concrete Ships was presented by H. C. Turner, its chairman.

President W. K. Hatt presented the special medal of the American Concrete Institute to Mr. W. Leslie Comyn, in recognition of his pioneer work in the development of the concrete ship "Faith." In the absence of Mr. Comyn the medal was accepted by Mr. H. C. Turner. The following papers were read and discussed:

"Construction of Concrete Barges for New York State Canal," S. C. Hollister.

"Layout and Equipment in Government Concrete Ship Yards," A. L. Bush.

"Problems in the Design of Concrete Ships," J. Glaettli, Jr.

PRESENTATION OF THE WASON AND COMYN MEDALS.

Four medals were presented at the 1919 convention, three of them the Wason Medal, for the years 1916, 1917 and 1918—which were not awarded during the war—and one a special medal from the Institute to Mr. W. Leslie Comyn, of San Francisco, in recognition of his work in building and promoting the first ocean-going American concrete ship, the "Faith."

The Wason Medals.

The awards of the Wason Medals—"for the most meritorious paper presented to the convention of the American Concrete Institute"—were as follows:

1916—A. B. McDaniel, "Influence of Temperature on the Strength of Concrete."

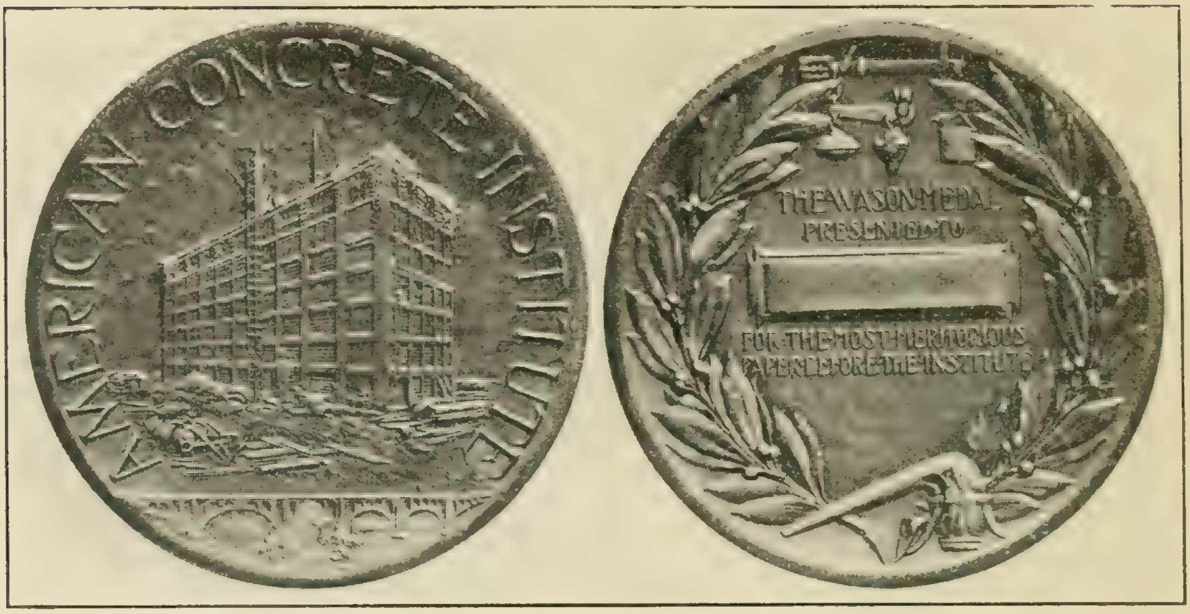
1917—Charles R. Gow, "History and Present Status of the Concrete Pile Industry."

1918—Duff A. Abrams, "Effect of Time of Mixing on the Strength of Concrete."

In the presentation of the award the following remarks were made:

President W. K. Hatt.—Some years ago a very strong and faithful man put the Institute on its feet. Having provided for the physical well-being of the Institute, he also gave thought to its spiritual welfare, and Mr. Wason at that time, in 1916, provided certain medals which were to be conferred upon those members of the Institute who presented papers of outstanding merit. Each year a certain paper was to be selected, and the writer of the paper was to be the recipient of the Wason medal. The Chair will ask Mr. Wason to present the medals.

Mr. L. C. Wason.—*Mr. President and Gentlemen:* Sentiment and the striving for an ideal are very vital forces in the affairs of mankind. If my memory serves me right, the first authentic written history referred to a contest for a prize, and the history and sentiment attached to the first Olympiads of the games which occurred many years before Christ and for a good many years which followed, have come down to our present time. The ideals for which they strove—physical prowess—at a time when physical prowess was very vital to the life of a small nation that had to defend itself against its neighbors, produced a sentiment which lasted to this day, so that the Olympiads were revived not many years ago. Today we know that mind is supreme. In the Holy Writ we have many references, for illustration, to the fact that mind is supreme and governs matter. It is, therefore, very fitting that a reward should be given to those who excel in mental attainments. While the early Greeks fought for a prize which was perishable, a laurel wreath, I believe, and strove against great odds and often at great sacrifice, so today we strive for a reward which is of little



THE WASON MEDAL.

SPECIAL MEDAL AWARDED BY THE INSTITUTE
to W. Leslie Comyn.

intrinsic value, but is connected with sentiment and seeks for an ideal which is worth while.

It ought to produce much better results in the affairs of the Institute and the industry it intends to serve, if this award is surrounded with such sentiment and ideals as to cause men to strive to obtain facts and then explain the truth. If the awards incite them to write out the truth of those facts in clear, concise, forceful manner for the benefit of their fellow-men, then they will be really worth while.

The rules which were adopted for the award of these prizes were not really original. They were merely compiled after studying the rules of the American and of the Boston Society of Civil Engineers, and I wish to inform the Board of Direction that if, in the future, conditions change and they find it wise and proper to change the rules, they are at perfect liberty to do so.

This award was made some years ago, but for certain reasons it seemed wise to postpone the production of them until now; therefore, there are three medals to award at this time. The Committee on Awards, with which I have nothing to do, selected the three men, and it is a great pleasure to me that they are all personal friends of mine, one of them a very dear friend, and it is with great pleasure that I award these prizes. On the face, each medal has a design which is typical of concrete construction; in the center, a reinforced-concrete building; at the bottom, an arch bridge, and at the margin, some unfinished work suggestive of construction. On the reverse side there is an olive wreath; at the top, a briquette suggestive of methods; and at the bottom, a retort suggestive of physical and chemical research and testing, and in the center, the inscription. The winner of the prize of 1916 is Prof. Allen B. McDaniel, and it is with great pleasure that I present him with this medal. The winner of the prize for 1917 is Charles R. Gow, of Boston, who, I believe, is not present, and I will see that he gets this medal, Mr. President, if you wish to leave it in my charge. The winner of the prize last year, 1918, is Prof. Duff A. Abrams, of Chicago. It is with great pleasure that I present it to him.

PROF. D. A. ABRAMS.—*Mr. Wason, Mr. President and Gentlemen of the Concrete Institute:* I am deeply conscious of the honor you have done me in presenting this medal. It seems to me that the American Concrete Institute is doing a very wonderful work in promoting the use and extensive knowledge of this material. I believe we are at the beginning of a new era in concrete work. It seems to me that the facts and papers presented before this meeting, notably such as those presented last night and today by Professor McMillan, Professor Slater and others of the Shipping Board, indicate a very marked widening in our knowledge with reference to this subject. In acknowledging my debt to the Institute, I want to express my indebtedness to the officials of the Portland Cement Association and to the trustees and faculty of the Lewis Institute, through whose coöperation in our construction material and research laboratory the carrying out of the experiments upon which this award is based was made possible.

MR. A. B. MCDANIEL.—Mr. Wason, I sincerely thank you for this award.

Mr. President and Gentlement of the Institute: Words can but inadequately express my appreciation of this honor which you have bestowed upon me. I shall be pleased to accept and treasure this medal, not only as a token of the accomplishment, which has seemed rather a small and simple thing to me, but principally as an emblem of the association of my colleagues, whose loyal coöperation made this work possible.

The profession and the Institute are to be congratulated upon the establishment of an institution of this character, which will doubtless be instrumental in promoting further investigations, research work and the publications of a notable character. Such a stimulating influence will certainly promote the progress of the profession and raise the standards of future papers to be presented before this society.

LT.-COL. C. R. GOW (*by letter*).—I wish to acknowledge receipt of the Wason Medal, which was awarded to me at the annual meeting of the Institute in June.

The selection of my paper on "Concrete Piles," for the award of this medal was a distinct surprise and source of gratification to me. Without such a complimentary acknowledgment, I should still have considered it a privilege to be permitted to contribute to the literature of the Proceedings of our society.

If the paper submitted by me contained matters of superior merit such as to warrant my selection for this signal honor, I feel doubly repaid for such effort as its preparation entailed.

I wish to thank you and the awarding committee most heartily for the distinguished consideration shown me and to express my regret that my military duties prevented me from being present to receive the award in person.

THE COMYN MEDAL.

The following is a report of the presentation exercises of the Comyn medal:

MR. S. E. THOMPSON (*from the Chair*).—One of our members has acplished so signal a service that the Institute deemed it its privilege to make recognition of this. Mr. W. Leslie Comyn is not able to be present here tonight, but Mr. H. C. Turner will act in his stead in receiving this recognition, and Dr. Hatt will make the presentation.

MR. W. K. HATT.—*Mr. Chairman and Members of the American Concrete Institute and Guests:* I think the instinct of most human beings is to trust and follow the man of faith. Faith is a quality which we are told will remove mountains, and our instinct is to follow him rather than the man of an analytical turn of mind. In the development of reinforced concrete, it was the men of faith who pushed ahead the early construction in advance of theory or codes or formulations. We honor our American pioneers who pushed through the trackless forest and wilderness to settle

the western country of the United States; and tonight the American Concrete Institute honors itself in recognizing the achievements of Mr. W. Leslie Comyn, of San Francisco, who, while others were thinking and talking of the difficulties and dangers of building and launching a concrete ship, assembled a group of San Francisco business men and constructed a concrete ship, the "Faith;" not a ship on a small or experimental scale, but a ship of large size, 5,000 tons. It was launched March 14, 1918, at Redwood City, California. Wisdom, it seems, was justified of her children, and this ship braved the storms of the ocean and made a trip around Cape Horn and brought her cargo in safety and herself in good structural condition to the port of the City of New York.

The American Concrete Institute, through its special Committee, of which Mr. H. C. Turner is Chairman, has had designed and cast a medal to commemorate this splendid achievement of Mr. Comyn, and presents it to him, through Mr. Turner, upon this occasion, with the homage of the Concrete Institute. This medal has on its two faces, first, a picture of the ship, "Faith," with a representation of the docks and some of the machinery used in constructing it, and on the rim the words "Concrete Steamship Faith, launched March 14, 1918, at Redwood City, Calif.;" and on the other face of the medal, "To W. Leslie Comyn. In Recognition of His Faith and Courage in Building the First American Ocean Going Concrete Ship. What Others Only Dreamed, He Dared To Do;" and on the rim the words, "American Concrete Institute." I take great pleasure, sir, in presenting this to Mr. W. Leslie Comyn, through his friend, Mr. Turner.

MR. H. C. TURNER.—Of course, you will appreciate that it will be impossible for me to express what sentiments Mr. Comyn might have on the occasion of the receipt of this medal from the Institute. I think it would only be proper for me to state that I have a telegram from him expressing his regret that he could not be present tonight and asking me to convey to the Institute his very deepest appreciation of the honor extended to him through the award of this medal.

Papers Read Before the 15th Annual Convention of the American Concrete Institute

STRUCTURAL LABORATORY INVESTIGATIONS IN REINFORCED CONCRETE MADE BY CONCRETE SHIP SECTION, EMERGENCY FLEET CORPORATION,

By W. A. Slater*

Early in the work of the Emergency Fleet Corporation on concrete ships the necessity for systematic investigation of certain of the problems involving structural action of reinforced-concrete members became evident. The work on the problems of this nature, which seemed to be of sufficient importance to justify some expense for investigation, and on which it was believed that laboratory tests would furnish useful information, was placed under the direction of the writer. The requirement of lightness as well as strength for concrete ships forbade the use of unnecessary material "in order to be on the safe side," and demanded exactness of design not ordinarily necessary. This resulted in the carrying out of a program of research in reinforced concrete which probably is more extensive than any other ever carried out by a single organization in a similar length of time in this or any other country. The extent of this work may be judged by the fact that there are on hand probably 4000 pages of original notes of the tests and by the fact that the amount of steel used for reinforcement will probably approximate 200 tons.

Some of the information obtained is of value only for use with reinforced-concrete ships, and, as is usual with such investigations, some of it will not be very useful even for this purpose. However, it seems safe to say that a large proportion of the results will be of much value not only in their application to the design of concrete ships but also in the broader fields of reinforced-concrete practice. Whatever loss there may be in making results of other lines of war activities serve useful purposes in peace-time needs, it seems certain that the value of this research work, in its effect on reinforced-concrete design, will be greater than its cost even at war prices.

The carrying out of such investigations successfully would not have been possible except for the coöperation of a number of organizations. The Office of Public Roads, Department of Agriculture, assisted freely by permitting the use of their laboratories and research organization. Lehigh University, Bethlehem, Pa., threw open the use of the Fritz Civil Engineering Laboratory for the use of the Concrete Ship Section. The laboratories of the Bureau of Standards at Pittsburgh and at Washington have been used freely for this work. The University of Illinois loaned testing apparatus and equipment and members of their investigational staff. Acknowledgment is made to these institutions for the assistance afforded.

Men of experience and high standing in investigational work were secured for the carrying out of this research program, and the value of the results depends as much upon this fact as upon the coöperation of other laboratories.

* U. S. Bureau of Standards, with Emergency Fleet Corporation, Philadelphia, Pa.

While the entire program was laid out in the office of the Concrete Ship Section under the direction of the writer, the men in charge of the various investigations were given as much freedom as possible in shaping the character of the investigation, the manner of carrying it out, and the manner of reporting on it. The size of the program made any other method impracticable and the writer believes it has been to the advantage of the work that this plan was followed.

Major A. R. Lord and Major W. M. Wilson, of the United States Army, A. T. Goldbeck, Engineer of Tests, Bureau of Public Roads, and G. A. Smith, of the Bureau of Standards, were in charge of different branches of the work. The following list gives the men who were employed by the Emergency Fleet Corporation for this work and the institutions from which they came. Acknowledgment is made to all these men for their hearty coöperation in making the investigational work successful:

B. A. Anderton, Bureau of Public Roads.
W. T. Bitting, Philadelphia.
J. O. Draffin, Ohio State University.
C. M. Ernst, Turner Construction Company.
M. O. Fuller, Lehigh University.
H. F. Gonnerman, University of Illinois.
H. P. Mueller, J. W. Mueller, Consulting Engr., Richmond, Indiana.
F. E. Richart, University of Illinois.
G. G. Scofield, Purdue University.
H. R. Thomas, Joint Committee on Stresses in R.R. Track.
R. R. Zipprodt, Joint Committee on Stresses in R.R. Track.

Acknowledgment is also made to R. J. Wig, Head of the Concrete Ship Section. Without Mr. Wig's recognition of the importance of an adequate research program and his vigorous support of the work, the investigations would have been less complete and the studies of the results would have been more superficial.

In the following pages are given some of the results most easily picked out for presentation from a number of the investigations. Several series of tests which have been made are not even mentioned in this presentation. Reports of all of these investigations are being prepared for publication by the Concrete Ship Section of the Emergency Fleet Corporation. In addition to this it is expected that the Bureau of Standards will extend certain of the investigations in directions most important and publish results for wider circulation than will be feasible for the Concrete Ship Section's report.

CORROSION TESTS.

The corrosion investigation was begun with the idea of determining the relative value of various kinds of coating which might be used for the purpose of protecting steel embedded in concrete from corroding due to the action of sea water and of sea air. Proprietary paints and bituminous coatings were used for the most part in this investigation. In addition, certain metallic coatings were used. Because the results were needed at the earliest possible

date, the salt-spray accelerated test was applied. No way is apparent of converting the length of time which specimens stood up under this severe test into actual time which they would stand up under service conditions. However, the results may be expected to show something of the value, relatively to each other, of various means of reducing corrosion. The various specimens consisted of 4×4-in. pieces of black iron of No. 16 gage coated with the material to be used as a protective covering. In the case of paints and bituminous coatings the specimens were allowed to harden as much as they would without long-continued exposure. This generally required from 5 to 15 days.

As a result of the salt spray tests it was found that any one of several different kinds of paints or bituminous coatings furnished a high degree of protection against corrosion. Their suitability for use depends on the practicability of applying a coating, of placing a coated bar of this kind in position without damaging the protective coating, and upon the fact that in all cases the bond resistance is seriously affected by such a coating.

Galvanizing and sherardizing protected the steel against corrosion, but a decomposition of the zinc itself took place. This raised the question of whether the salt which forms in this decomposition would have the same effect in splitting the concrete away from the bar as occurs when the unprotected bars are used. Electrolytic tests, in which both galvanized and unprotected bars were used, showed that the galvanized bars split due to corrosion within a shorter time than did the unprotected bars. There was uncertainty, however, as to whether the same action would occur in corrosion which was not due to electrolysis.

A proprietary method of protecting steel from corrosion which involves a chemical action of a phosphorus compound on the steel was tried. This gave poorer resistance to corrosion than any of the paint or bituminous coatings. However, when compared with unprotected steel there was sufficient retardation of corrosion to indicate that this method of protection may have decided value for treatment of steel which is embedded in concrete, since it caused no apparent loss in bond resistance.

The results of the tests made were not sufficiently conclusive to warrant the adoption of any anti-corrosion treatment for the reinforcement in the concrete ships which are under way. However, along with the salt spray tests, other methods of accelerating corrosion and of measuring the amount of corrosion were studied with the expectation of carrying out an investigation of this kind.

Important information was developed which indicated that the corrosion of steel embedded in concrete could be accelerated by the use of an atmosphere of oxygen, carbon dioxide and steam as a corroding agent. It seems necessary that this be introduced under pressure in order to penetrate the concrete. Various solvents for the corroded steel were tried in the effort to find one which would clean the specimen of all corrosion without removing any of the metallic steel. With such a solvent it is possible to determine the amount of loss of weight by corrosion.

It will be obvious that the solvent which removes the corrosion in the

shortest time and after that has the least effect on the metal is most suitable for the purpose required. Figs. 1 and 2 show the results of this investigation. Cold saturated boric acid seems to be one of the most suitable reagents for this use.

An investigation was to have included corrosion tests made in the manner

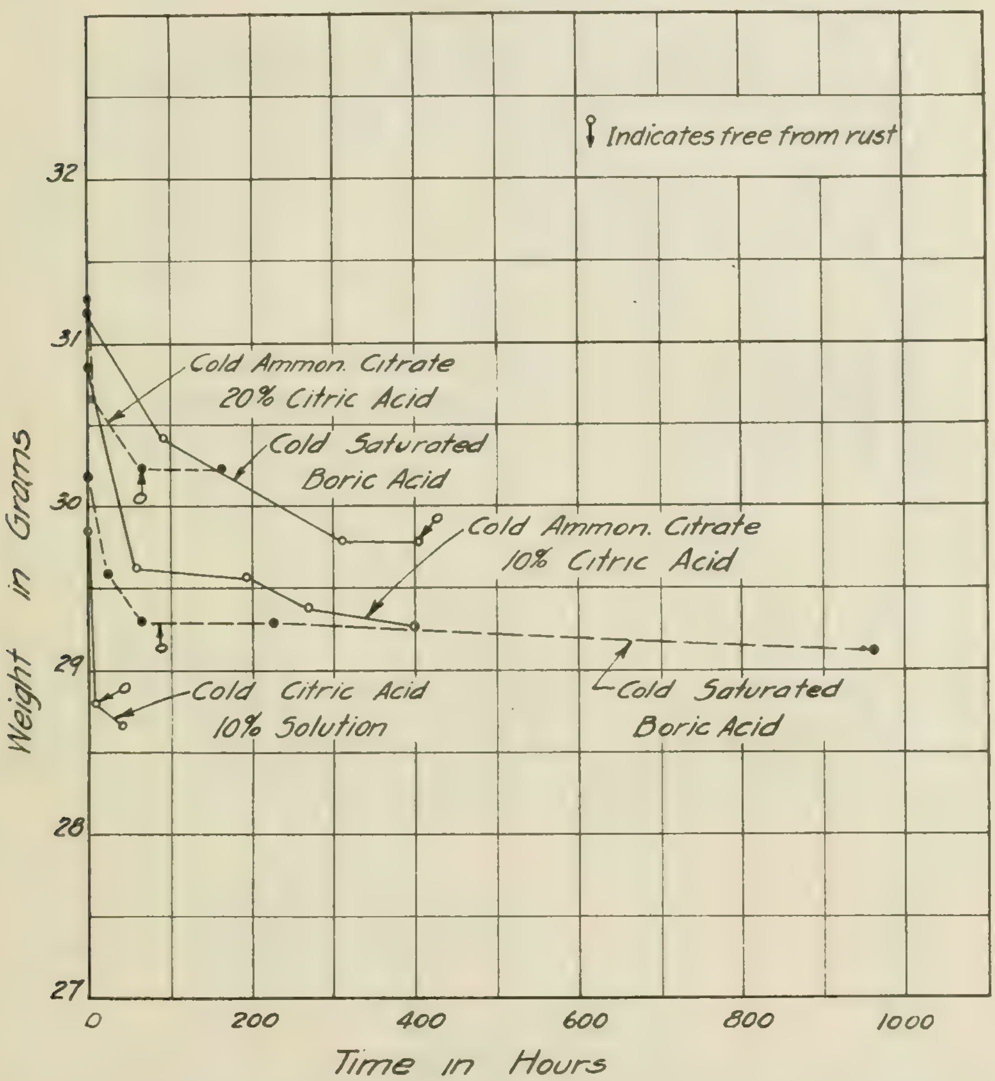


FIG. 1.—RATE OF REMOVAL OF CORROSION BY VARIOUS SOLVENTS APPLIED COLD.

here outlined, but the work did not reach this stage. It is planned to carry out such an investigation in the near future at the Bureau of Standards.

LEAKAGE OF WATER THROUGH CRACKS IN CONCRETE WALLS.

Early in the investigation of shearing strength of reinforced-concrete beams it was found that cracks were likely to form when shearing stresses were considerably lower than those which it seemed necessary to use in the

design of concrete ships; yet it was not known whether these cracks were such as would permit leakage of water through them or not. Tests were made in which hollow beams were loaded in such a way as to cause diagonal tensions cracks in the side walls at the same time that the beam was filled with

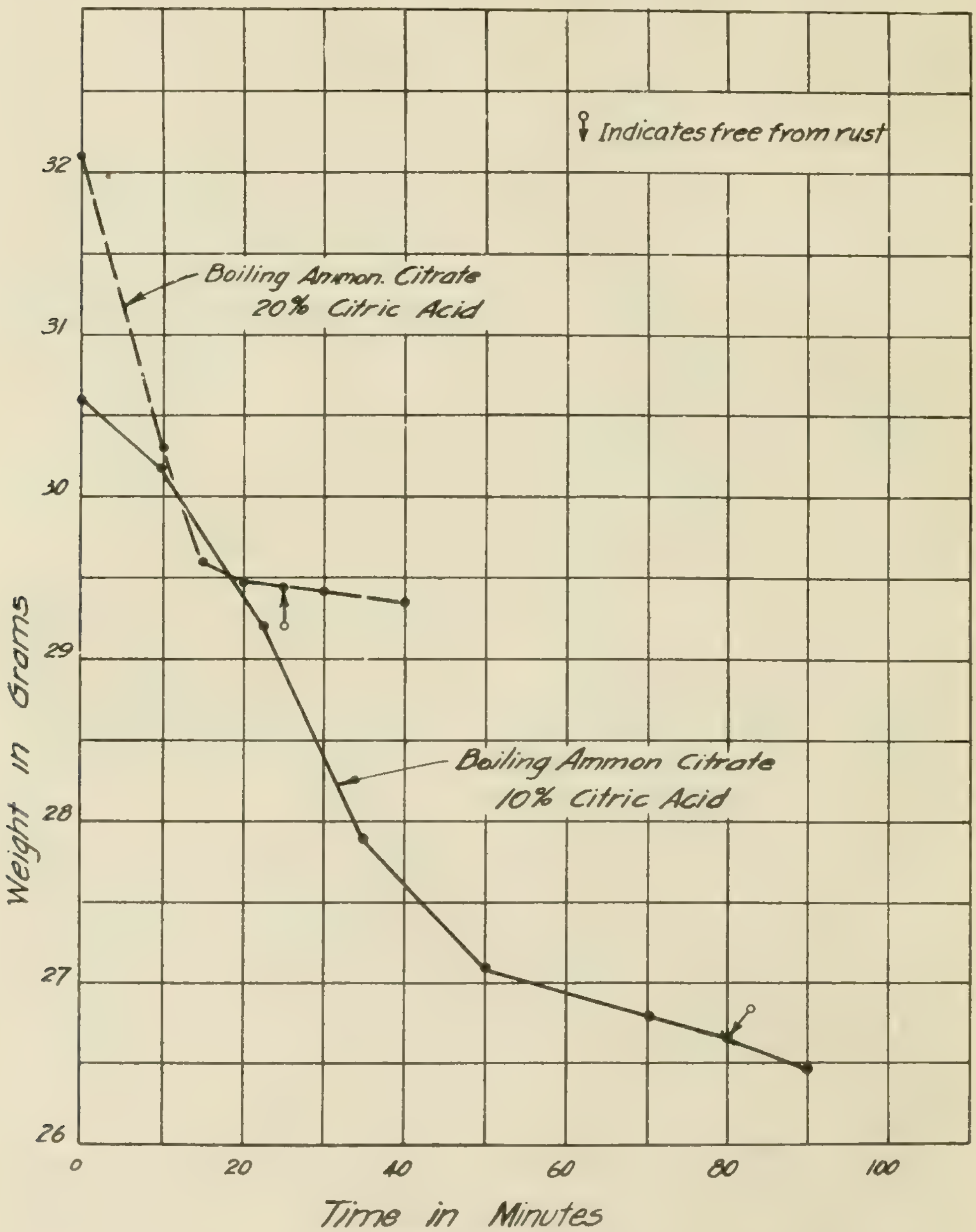


FIG. 2.—RATE OF REMOVAL OF CORROSION BY VARIOUS SOLVENTS APPLIED HOT.

water which was maintained under pressure varying in head from 15 to 30 ft. These tests indicated that the smallest crack which could be detected, say 0.001 in. or less, in width, would permit the passage of enough water to cause a moist surface on the outside around the crack. As the crack width increased,

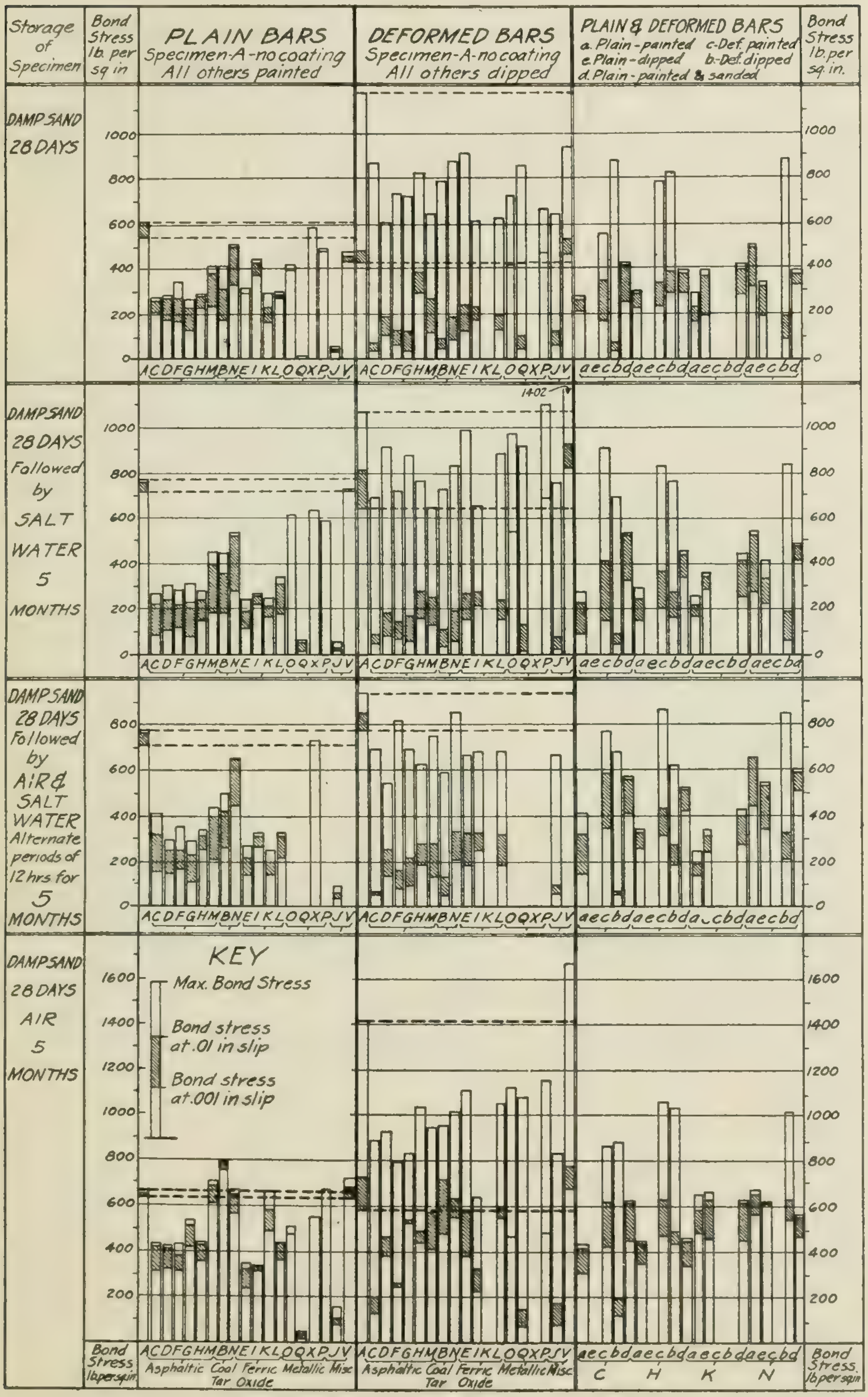


FIG. 3.—CHART SHOWING EFFECT OF ANTI-CORROSION COATINGS ON BOND STRENGTH.

the leakage increased rapidly, provided that time was not given for the closing up of cracks by the deposit of sediment of any kind. When the crack was 0.01 in. wide, water spurted clear of the outer surface of the beam.

When the beam was allowed to stand over night with the pressure head and the load maintained, the leakage was decreased markedly. This was probably due to the deposit of a substance which helped to fill up the cracks. Further indication of this was shown by the appearance of efflorescence on the surface of the beam in the neighborhood of the cracks.

If the cracks had been very small at the time of their formation and had developed very slowly, it is possible that the silting would have kept up with

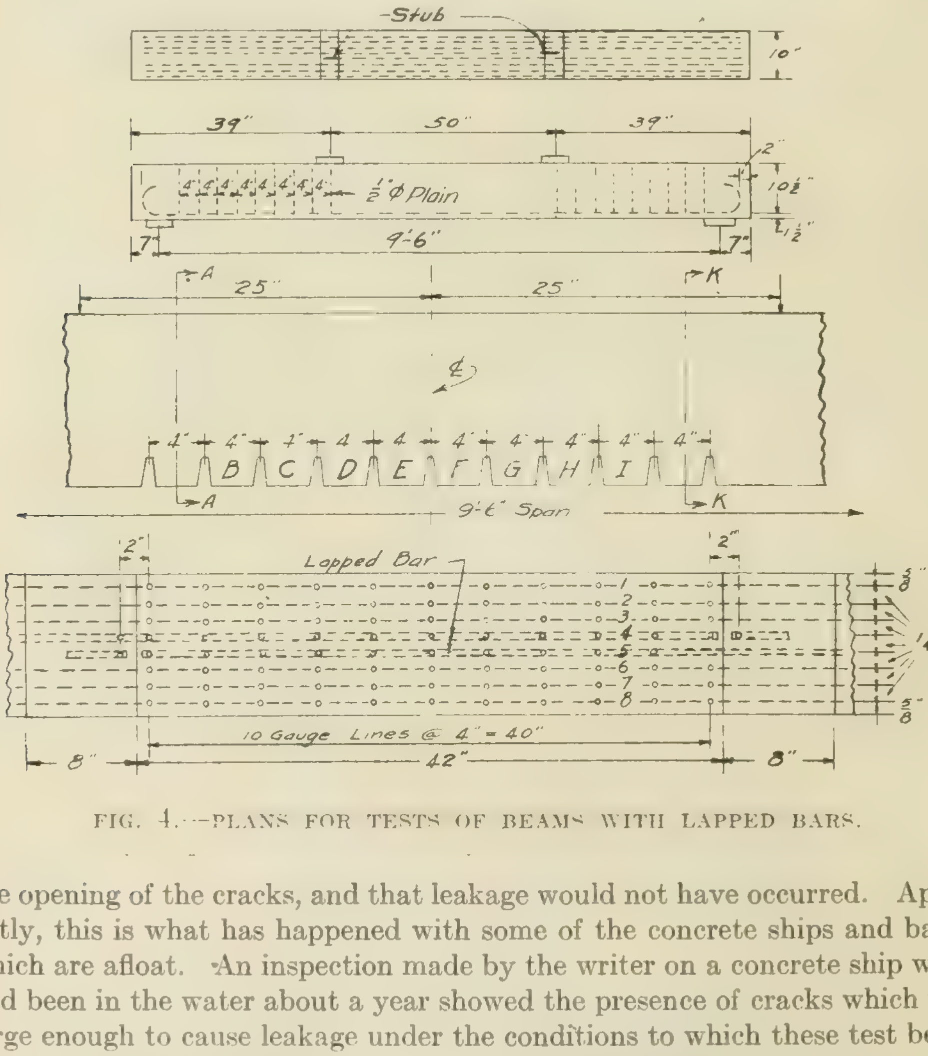


FIG. 4.—PLANS FOR TESTS OF BEAMS WITH LAPPED BARS.

the opening of the cracks, and that leakage would not have occurred. Apparently, this is what has happened with some of the concrete ships and barges which are afloat. An inspection made by the writer on a concrete ship which had been in the water about a year showed the presence of cracks which were large enough to cause leakage under the conditions to which these test beams were subjected. However, there was no leakage, although the cracks were below the water line and apparently there had been none. There was efflorescence apparent at cracks on the inside of the hull, indicating that the process of silting had been taking place. To the writer this seems more nearly a conclusive demonstration of the ability of the concrete ship to stand up under the conditions of service than would be the case if there were no cracks, and if the

possibility of cracks yet forming were present, together with uncertainty as to whether they would close up due to formation of silt.

BOND TESTS OF COATED BARS.

Bond tests were made to determine the effect of the various anti-corrosion treatments on the bond resistance of a bar. These were pull-out tests in which the specimens consisted of 6×6-in. concrete cylinders with ½-in. square bars, both plain and deformed, embedded axially in the cylinder. The amount of slip was measured accurately. The results of these tests indicate that in

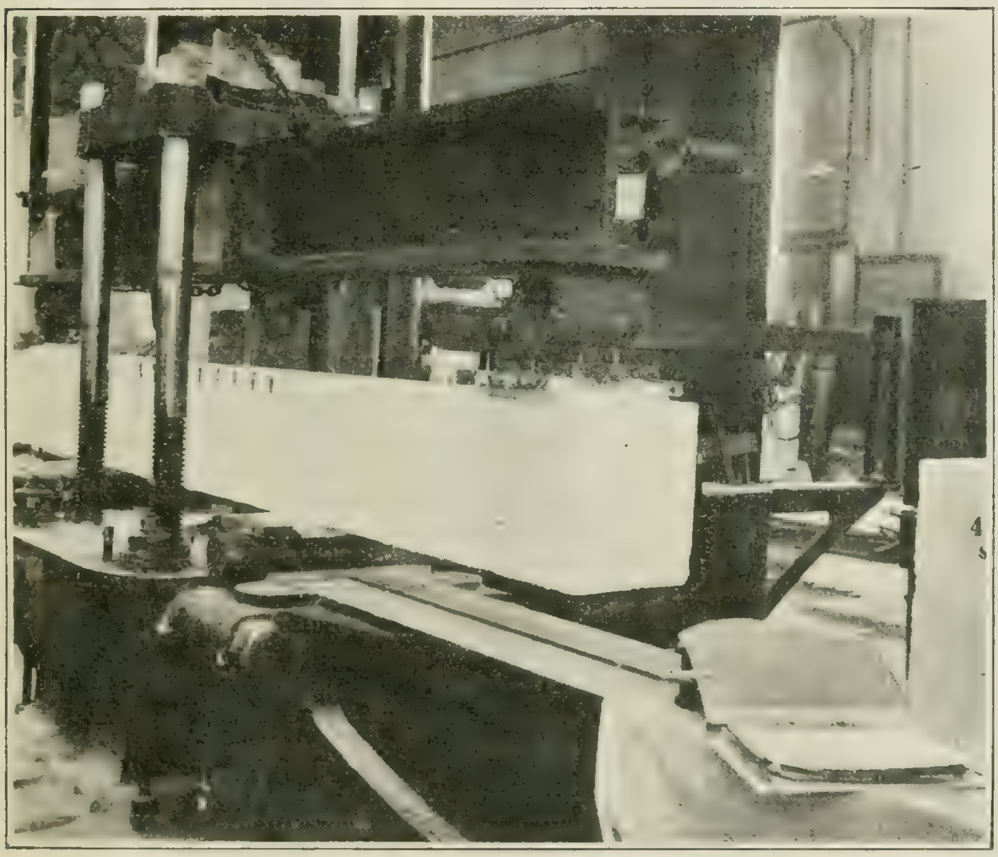


FIG. 5.—VIEW OF BEAM WITH LAPPED BARS IN TESTING MACHINE.

general all protective treatments used, except the phosphate treatment, reduced the bond resistance materially. This statement is true, regardless of whether plain bars or deformed were used, and regardless of the amount of slip which is to be taken as the basis for comparison. Fig. 3 is a chart which gives a summary of the results obtained with various treatments.

BOND TESTS WITH LAPPED BARS.

General Features.—Beams were made in which a ½-in. bar was lapped over a distance of 40 in. at the center of the span. In one beam no other

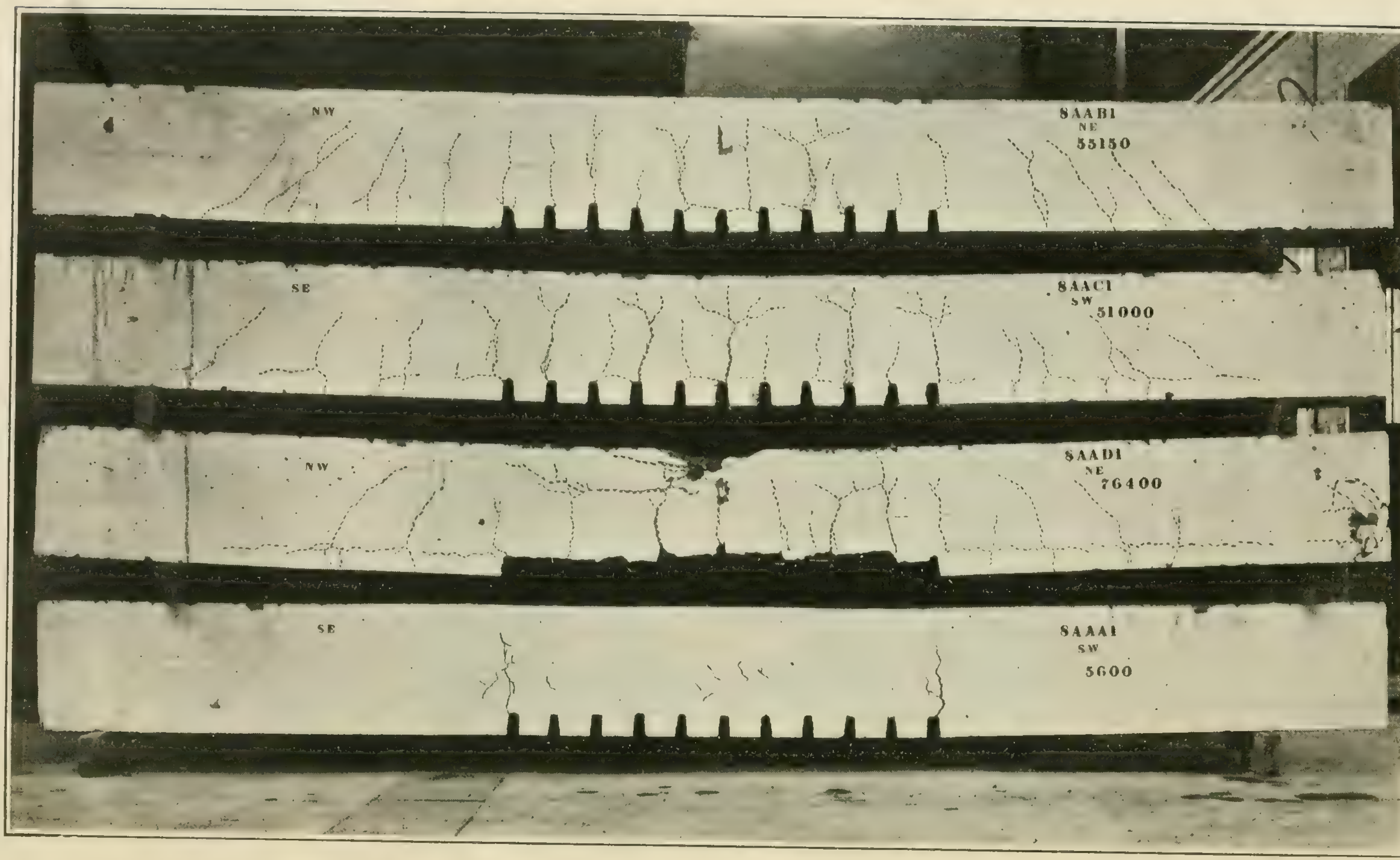


FIG. 6.—APPEARANCE OF BEAMS WITH LAPPED BARS AFTER TEST.

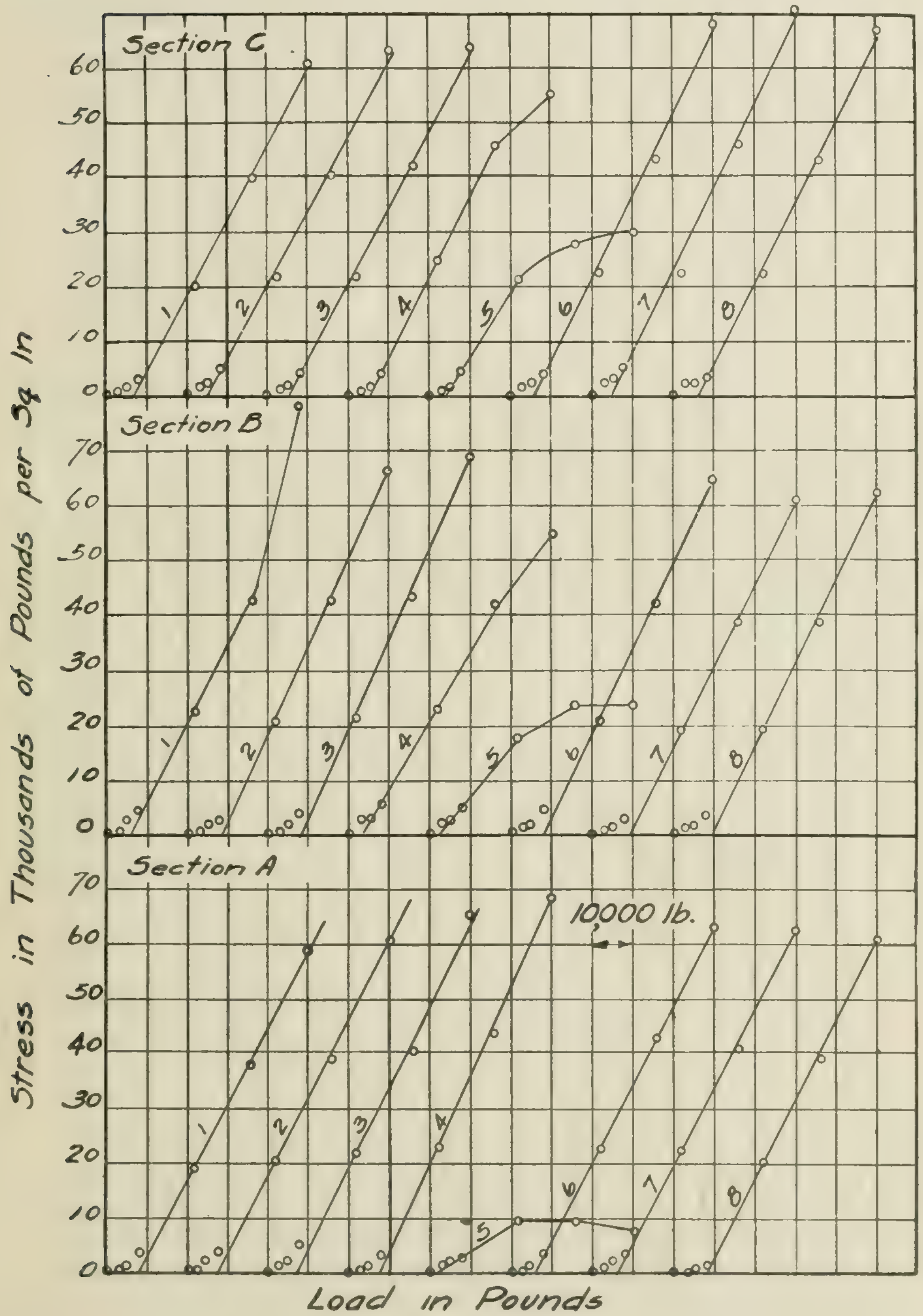


FIG. 7.—LOAD STRESS GRAPHS FOR BEAM 8AAB1; SECTIONS A, B AND C.

reinforcing bars were present. In all others six additional bars were used. The size of the additional bars was $\frac{1}{2}$ in. in one beam, $\frac{5}{8}$ in. in another, and $\frac{3}{4}$ in. in another. Only these four beams were tested. The concrete used was of a mix having the proportions of about 1 part cement, $\frac{2}{3}$ parts fine aggregate, and $1\frac{1}{3}$ parts coarse aggregate in which the largest pebble passed a $\frac{1}{2}$-in. screen. Measurements of stress were taken at various points along the lapped bar, as

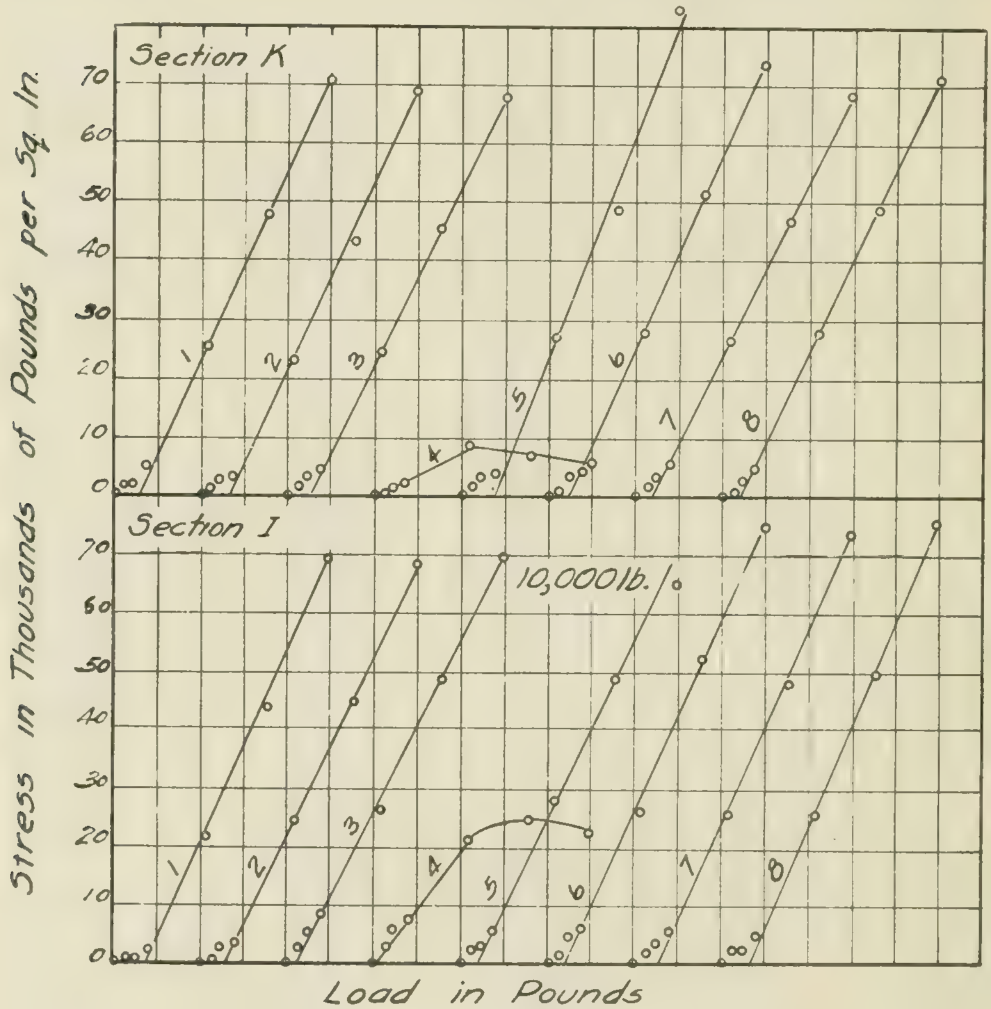


FIG. 8.—LOAD STRESS GRAPHS FOR BEAM 8AAB1; SECTIONS I AND K.

well as along the through-bars. The position of the laps and the location of gage lines for the measurement of stresses along the bars are shown in Fig. 4.

Fig. 5 shows a beam in the testing machine. The notches at right angles to the axis of the beam were necessary in order to expose the bars for measurement of deformation. The bond resistance probably was slightly decreased by the presence of these notches. Fig. 6 shows the appearance of the beams after test. In beam 8AAD1 it will be noted that there was a horizontal crack at the elevation of the reinforcing bars nearly the full length of the beam, and that at the right-hand end there was cracking under the semi-circular

hook used to anchor the bar. This probably was due to the fact that the bar was very close to the surface of the beam, but it indicates that there must have been a partial failure in bond in order to bring the hooks into action. At

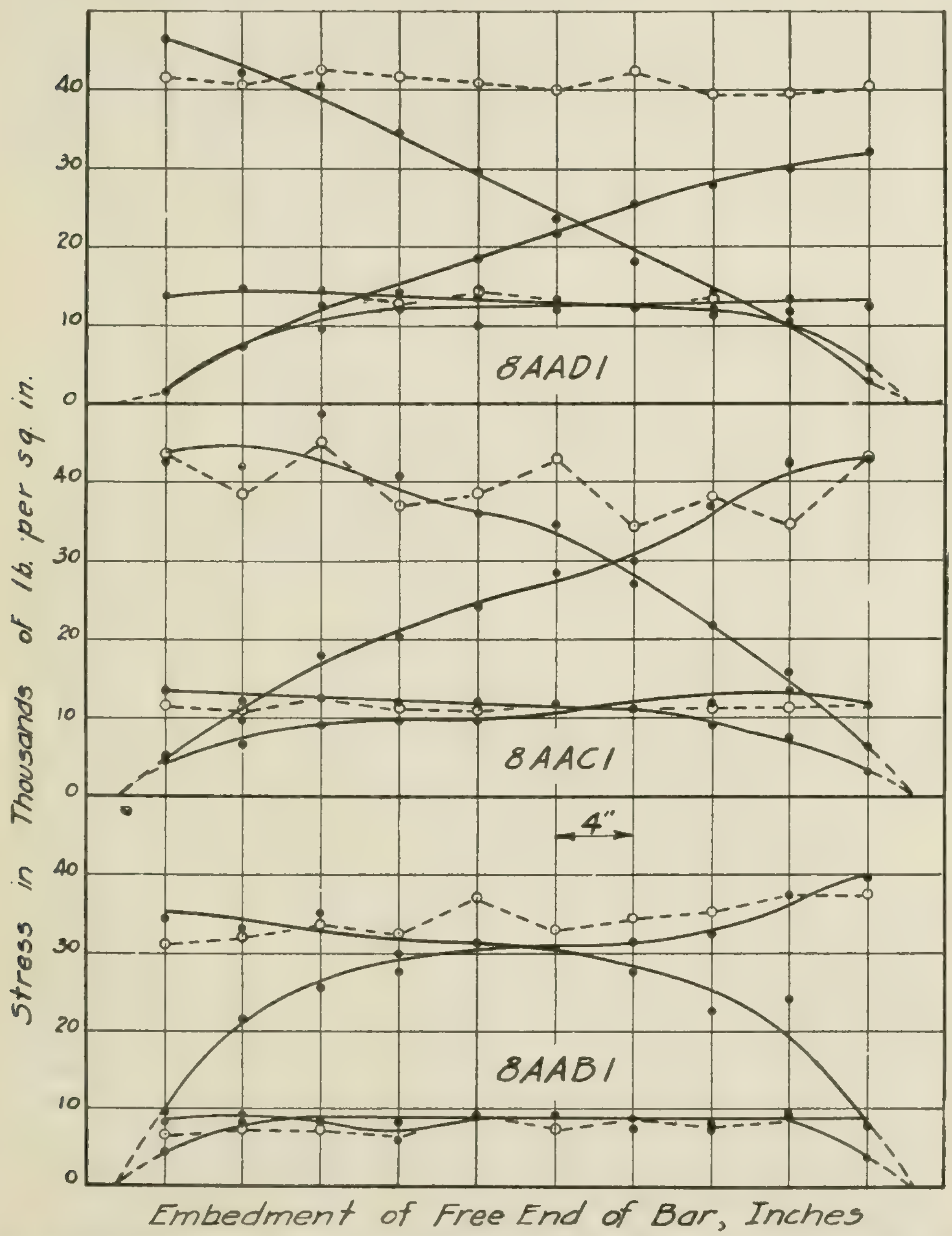


FIG. 9.—DISTRIBUTION OF TENSILE STRESS ALONG REINFORCING BARS FOR LOADS CAUSING COMPUTED STRESSES OF 16,000 AND 40,000 LB. PER SQ. IN.

the maximum load the computed bond stress for this beam was only 270 lb. per sq. in. In judging of proper working stresses in bond, an end slip of 0.01 in. in pull-out tests has sometimes been used as the criterion. Reference to Fig. 3

shows that for pull-out tests using a concrete not less strong than that in these tests the bond resistance for uncoated bars at 0.01 in. end slip was 600 lb. per sq. in. The results of this test indicate that the factor of safety for unanchored bars is less than would appear if the working bond stress is based upon the resistance when the end slip is 0.01 in. Figs. 7 and 8 show the load-stress graphs for all the bars of beam 8AAB1 at Sections A, B, C, I, and K. Bars 4 and 5 were lapped with bar 5 ending near Section A, and 4 ending near K. Fig. 7 shows (by the smallness of the stress) that at Section A, bar No. 5 was of little use in resisting the bending moment and that it increased in effectiveness from section to section until at Section K (see Fig. 8) it was fully as effective as any bar in the beam. Likewise bar No. 4 was ineffective at section K, but at section A was fully effective. The distribution of the

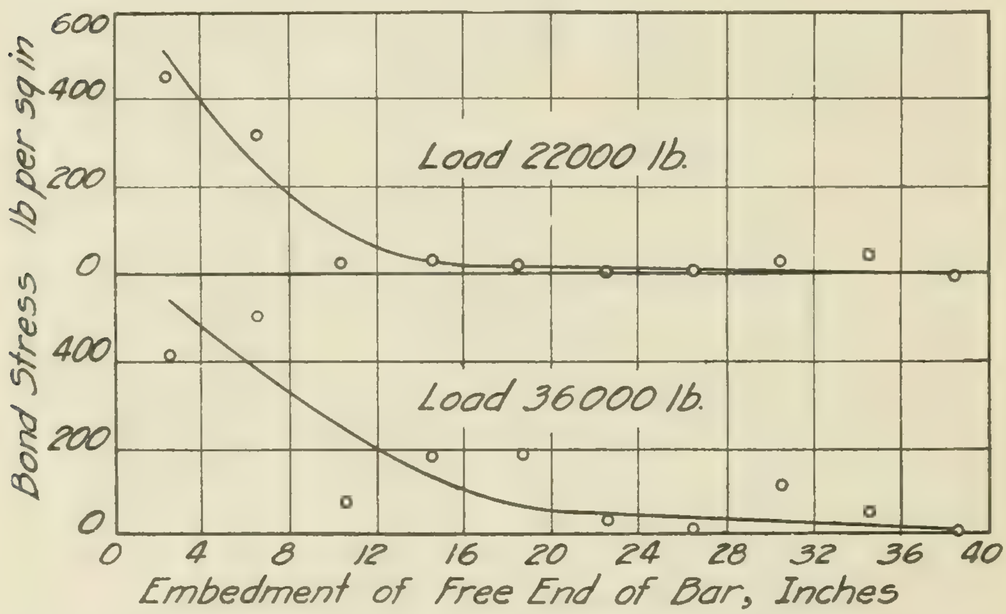


FIG. 10.—DISTRIBUTION OF BOND STRESS ALONG BARS OF BEAM 8AAB1.

stresses along the bar is shown in Fig. 9 for beams 8AAB1, 8AAC1, and 8AAD1. The solid lines show the stresses in the lapped bars and the dotted lines show the average for all other bars in the beam. The curves are given for two different loads for each beam. These loads are those which will give computed tensile stresses of 16,000 and 40,000 lb. per sq. in. in the reinforcement. From the slopes of these curves the bond stresses may be determined. These are shown in Fig. 10 for beam 8AAB1 for loads of 22,000 and 36,000 lb. The distribution of points on curves for Figs. 9 and 10 is somewhat erratic, due to the formation of cracks which will cause tensile stresses and bond stresses to be set up which are not in accordance with the assumption of perfect bond resistance. The amount of slip for beam 8AAB1 at various points along the reinforcing bars is shown in Fig. 11 for loads of 22,000 and 36,000 lb. The greater regularity for these points is due to the fact that they represent the

summation of deformations along a bar. By this process of summation the irregularities due to cracks are removed to a great extent.

Length of Lap Required for Beams with Through-Bars.—To serve as a working basis it may be assumed that for any given tensile stress the safe length of lap is the distance from the free end of the bar to the point where the tensile stress in the lapped bar becomes constant and equal to the stress in an adjacent through-bar. Analysis shows that if the lap furnished is less than this the tensile stresses in the through-bars at the ends of the lap will be greater than the value computed by the equation:

$$f_s = M/A_s\, jd$$

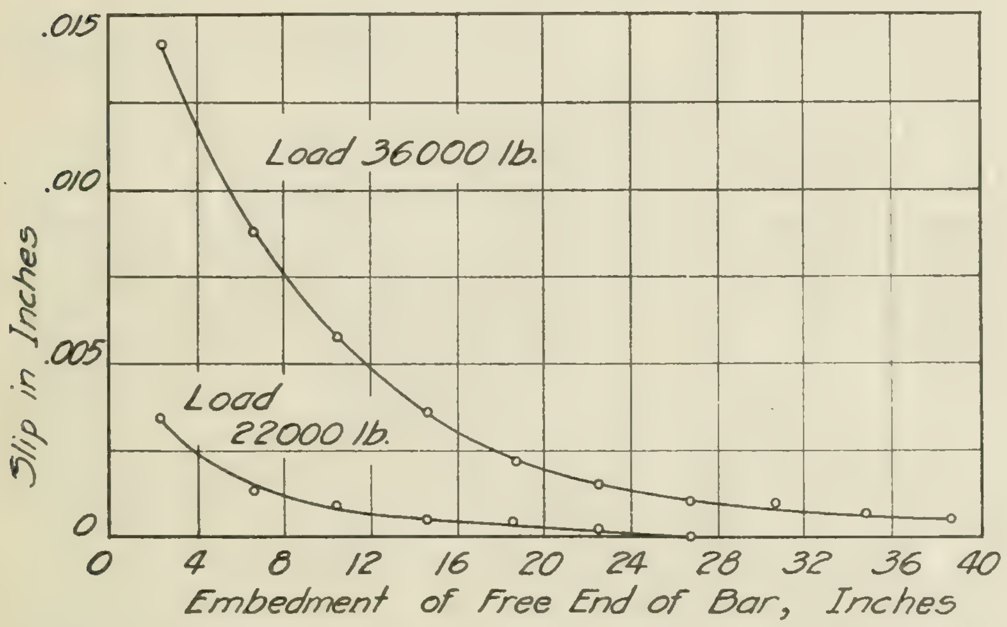


FIG. 11.—SLIP AT SUCCESSIVE POINTS ALONG REINFORCING BARS OF BEAM 8AAB1.

This is based on the assumption that the entire lap lies within a region of constant bending moment.

With this assumption as a basis, Fig. 12 has been prepared. The data for the beam without any through-bars were not used in this figure. The amount of reinforcement for this beam was very small; consequently, the concrete took a large portion of the stress, and the stress curves for the steel were too erratic for use in this way. The ordinates of the points given represent the distance from the ends of the lapped bars at which the measured stresses shown as abscissas were developed. A straight line represents fairly well the mean of the values shown by the points. The equation of this line is:

(1) $s = .00059\, f_s$, where

s = distance in inches from end of lapped bar to point where bond stress becomes zero, and

f_s = the tensile unit stress under consideration.

It is apparent that the total tensile stress in a lapped bar at the section where the bond stress becomes zero is:

(2) $\frac{\pi a^2 f_s}{4} = \pi aus$, where

a = diameter of bar, and

u = average bond stress in distance s.

From this equation

(3) $f_s = \frac{4\ us}{a}$

Substituting in equation (1).

(4) $s = .00059\ \frac{4\ us}{a}$

(5) $u = \frac{a}{4 \times .00059} = 210$ lb. per sq. in.

This indicates that the value of the average bond stress was independent of the tensile stress developed. The adoption of this form of equation is equivalent to concluding that the average bond stress over the length s was the same at all stages of the test, but that the length s increased as the tensile stress increased. Reference to beam 8AAB1, Fig. 10, shows that the distance s from the end of the bar at which the bond stress becomes zero was greater for the load of 36,000 lb. than for the load of 22,000 lb. but that the average bond stress was not far from 210 lb. per sq. in. in both cases. For the other beams the average bond stress was less than this. It probably is not exactly true that the average bond stress developed at the ends of lapped bars is independent of the tensile stress developed, but as a basis for design this assumption is probably exact enough.

It is generally assumed that the length of embedment required to develop the tensile strength of the bar is directly proportional to the diameter of the bar. In this investigation it was not feasible to test a sufficient number of beams with lapped bars to check this, but it seems a logical assumption to make. In these tests $\frac{1}{2}$-in. bars were used. That is

(6) $a = \frac{1}{2}$.

Dividing equation (1) by equation (6) gives an equation for the number of diameters of lap required to develop the stress f_s.

(7) $\frac{s}{a} = \frac{.00059\ f_s}{\frac{1}{2}} = .00118\ f_s$

The stress in the lapped bar just outside the lap is the f_s of this equation. If this is less than the average stress at this section calculated by the equation

$$f_s = \frac{M}{A_s\ jd}$$

the stress in the through-bars must be greater. Consequently, in using this equation to state the safe length of lap, the value of f_s should be the yield-

point stress. Otherwise, the yield-point stress will be passed in the through-bars before reaching it in the lapped bars. Putting the yield-point stress at 40,000 lb. per sq. in.

(8) $\frac{s}{a}=0.00118\times40{,}000$

(9) $s=47.2a$

The concrete used in these beams had a cylinder strength of about 5,000 lb. per sq. in. and for leaner concrete it is to be expected that a greater lap would be required.

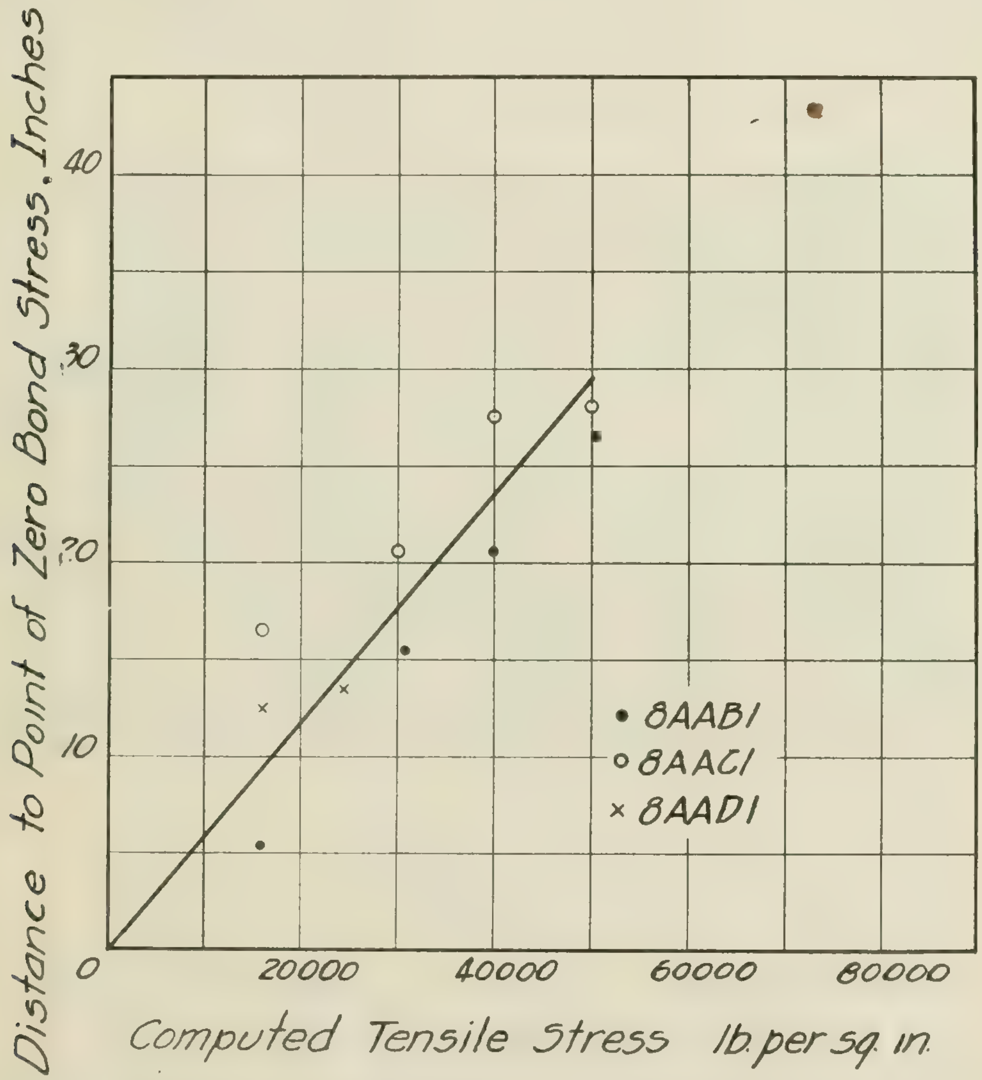


FIG. 12.—RELATION OF TENSILE STRESS IN REINFORCEMENT TO LENGTH OF LAP REQUIRED.

This may seem a large requirement for the lap, in view of the fact that an embedment of 40 diameters is generally supposed to be sufficient to develop the full strength of a bar even with a leaner concrete than that used here.

However, it must be remembered that this opinion is based generally upon pull-out tests of specimens which have no tension in the concrete surrounding the bar. In a region of high tensile stress considerable cracking of the concrete

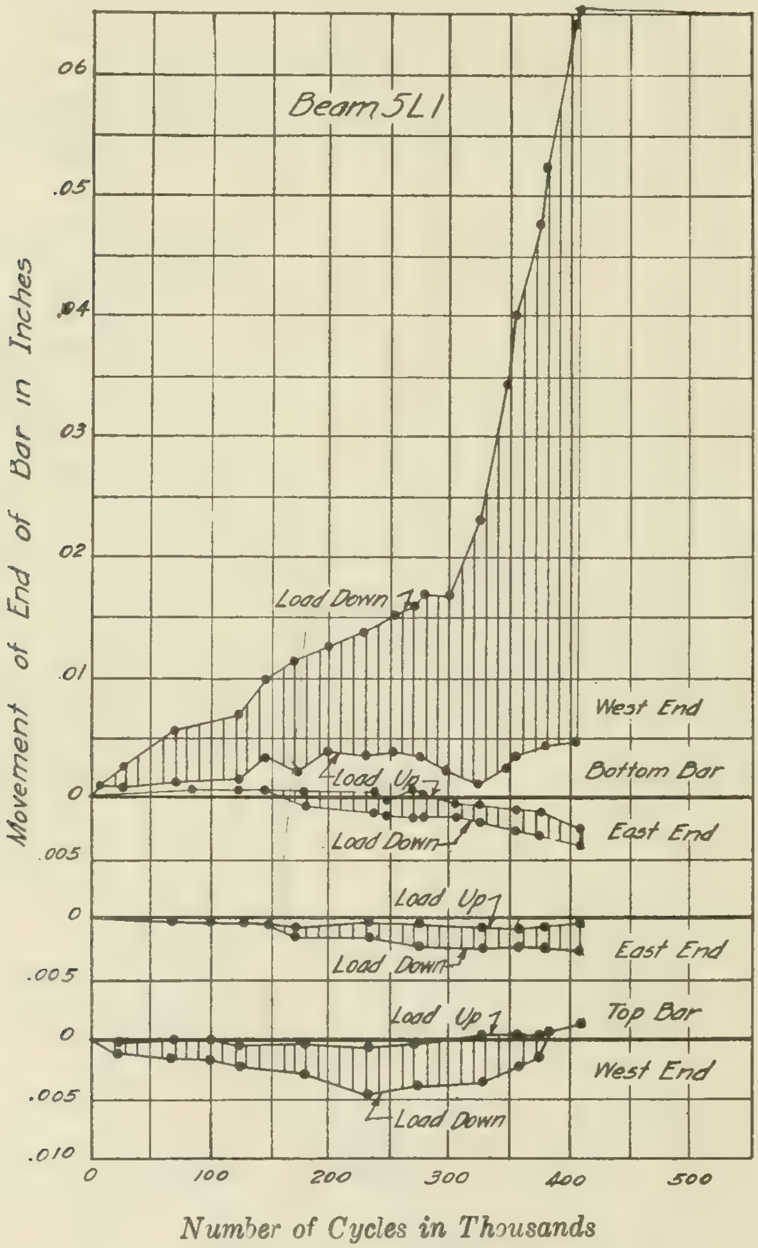


FIG. 13.—SLIP OF BARS IN BEAM 5L1 UNDER REPEATED REVERSAL OF LOADING.

may occur. This tends to destroy bond resistance and a greater length of embedment would be expected to be necessary.

Furthermore, as pointed out in a previous paragraph, there were indications that in beam 8AAD1 there was some slip over the entire length of the bars, although the bond stress developed was only 270 lb. per sq. in.

REVERSAL OF STRESS.

One of the earliest questions to arise in connection with design of reinforced-concrete ships was that of the deterioration of concrete due to a grinding and abrasion of the fractured surface of concrete on the tension side of a structure subjected to reversal of stress. Tests were made on concrete beams of a 1 : 2 mix to ascertain how serious this action might prove to be. The beams were 6 in. wide, 8 in. deep, and 8 ft. long, reinforced longitudinally in the top and in the bottom. They were subjected to reversal of loading at the rate of about seventeen cycles (17 loads up and 17 loads down) of loading per minute. At the present time, only four specimens have been tested to failure. Of these all have failed by tension in the reinforcement at a much smaller number of applications of load than is generally expected for tests of

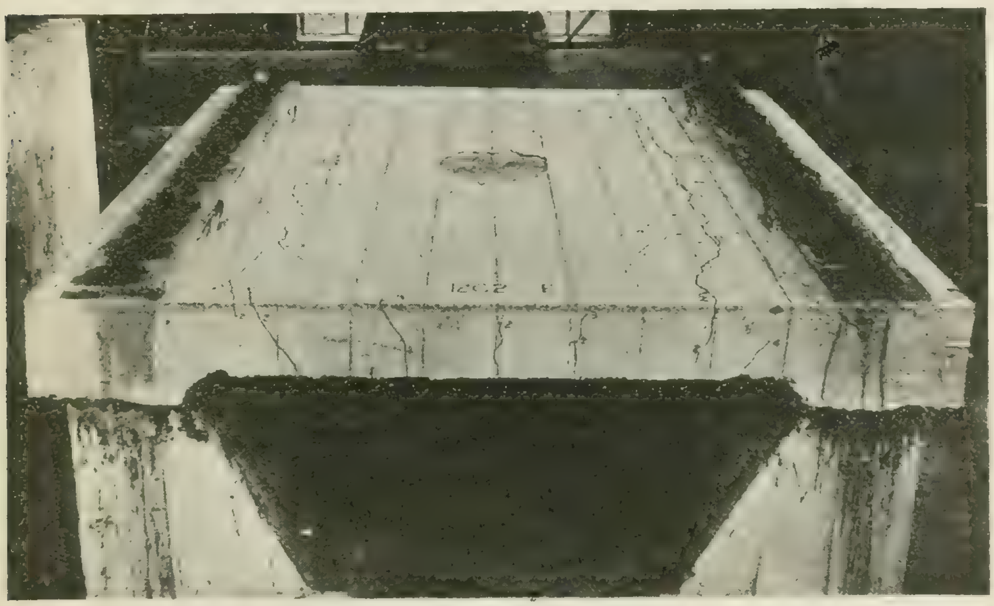


FIG. 14.—UPPER SURFACE OF SLAB 12C2 AFTER 24-IN. BLOW.

steel. There are a number of features of the test which may be considered partial explanations of the failure but none of them are entirely satisfactory.

In one beam, which had $1\frac{1}{4}$-in. bars without mechanical anchorage, less than 0.001 in. slip could be detected within the first 7200 applications of load. At this point slipping began and increased in amount until at 400,000 applications the slip was more than 0.06 in. In Fig. 13 the amount of slip which took place with each application is shown as the shaded area between the two curves representing the conditions for the upward load and the downward load. Failure by slipping was expected and probably would have occurred within a short time except for the fact that tension failure occurred first.

One of the beams was made I-shaped in cross-section in order to introduce high shearing stresses. This beam proved to be very tough. The web

reinforcement consisted of vertical stirrups spaced 2 in. apart welded to the reinforcing bars at the top and at the bottom.

The grinding of the fractured surfaces of the concrete was found to be negligible within the range of tests completed. In one of the beams after about 600,000 applications of stresses of 1550 lb. per sq. in. in compression in the concrete and 20,000 lb. per sq. in. tension in the steel, there was a slight indication of grinding and abrasion of the fractured surface, but other circum-

FIG. 15.—UNDER SURFACE OF SLAB 12C2 AFTER 24-IN. BLOW.

stances connected with the test indicated that this condition was only at the surface of the beam and did not extend to its interior. The stress observations indicated that throughout the test the beam was acting normally and that abrasion of the fractured surfaces did not affect in any way the amount of resistance afforded by the concrete to the compressive stresses in the beam.

For a beam subjected to 2,000,000 applications of a load which caused measured deformations corresponding to tensile stress in the reinforcement of 11,000 lb. per sq. in. and a compressive stress in the concrete of about 1450

lb. per sq. in., the largest crack measured 0.003 in. For this beam no indication is present of any grinding action between the fractured surfaces of the concrete and there is nothing to indicate that the beam is approaching failure.

These tests have developed failures which are difficult to explain but which point out the importance of careful inspection of reinforcement for a structure which has to undergo frequent repetition of stress.

FIG. 16.—UPPER SURFACE OF SLAB 12H2 AFTER 66-IN. BLOW.

EFFECT OF IMPACT ON CONCRETE SHELLS.

The test to determine the resistance of concrete to impact was difficult to devise. The best which seemed to be available was a comparison between the resistance of a concrete shell of the thickness used in concrete ships and that of a steel plate of the thickness used in steel ships of the same cargo-carrying capacity as the concrete ship. It is recognized that the effect of impact would show up in a different way on a concrete shell from that on a steel shell. The concrete shell is weak in diagonal tension, but deflects little

under a given load. The steel shell would show a large deflection relatively to a concrete shell but would have no weakness in diagonal tension. As a result, failure in the concrete shell should be expected by diagonal tension and in a steel shell probably by ripping of rivet seams. In order to make the tests comparable it was necessary to surround the concrete slab with heavy frames corresponding to those in the ship and to restrain the steel slab in such a way as to resemble the restraint afforded by the plates adjacent to the posi-

FIG. 17.—UNDER SURFACE OF SLAB 12H2 AFTER 66-IN. BLOW.

tion where impact occurred. An attempt was made to design the test pieces in such a way that these restraints would be introduced, but it is not certain that this was accomplished fully.

It was recognized that design could not be expected to provide against impact such as would occur in a collision between two ships, and that the most which it was practicable to furnish in either a steel ship or a concrete ship was sufficient strength to resist the impact of a small object such as a

log floating in water. With this in view, a testing apparatus was designed which consisted of (a) a heavy base to afford support and restraint for the test specimen and with sufficient mass to absorb considerable of the energy of the impact, without the movement of the base being appreciable; (b) a spherical ball of cast iron weighing 2000 lb. to be dropped upon the slab to furnish the impact and (c) a device for measuring the vibration of the slab due to the impact.

Some of the tests were made with the cast-iron ball dropping directly on the surface of the slab and others with a cushion of oak 6 in. thick and 12 in. in diameter interposed to break the force of the blow. The latter seems to represent more nearly the possible conditions of impact against a ship in service than the case in which no protection is offered.

Two slabs of each kind were tested. In the first test of each kind of

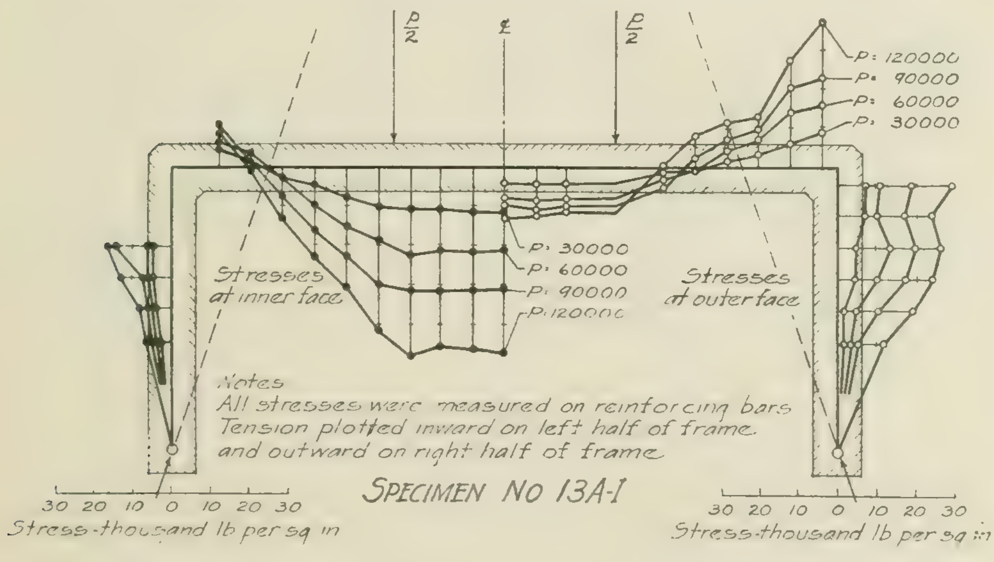


FIG. 18.—OBSERVED STRESSES IN FRAME 13A1.

slab the height of the fall was varied beginning with 6 in. and increasing by 6-in. increments until failure occurred. Later tests varied this procedure by making the height of fall at the first application as much as was believed to be necessary to cause failure. It was found that with the latter procedure the slabs showed much greater resistance to impact than with the former.

Fig. 14 shows the appearance of the upper surface of a 4-in. slab 6 ft. 6 in. by 7 ft. The span was 5 ft. 3 in. The reinforcement consisted of ½-in. bars placed 2¼ in. on centers in each diagonal direction. Fig. 15 shows the appearance of the bottom of the same slab. Slab 12H2, shown in Figs. 16 and 17, was of the same thickness, but was 10 ft. by 10 ft. 4 in. in size, and had a span of 9 ft. 2 in. between the centers of bolt holes. The weight of reinforcing steel was the same as for 12C2, shown in Figs. 14 and 15, but the bars were placed parallel to the sides. There was some difference in the proportional distribution of steel between the two layers for the two slabs.

In his report on the impact tests Professor H. F. Gonnerman, who was in charge of the investigation, draws conclusions which are about as follows:

"The thickness of the slab has considerable effect on the resistance to impact, the greater thickness having the greater resistance." The force of the blow required to cause failure appears to be approximately in direct proportion to the depth of the slab from the compression surface to the center of the tension reinforcement.

The resistance to impact increased as the span increased. The effect of variation of span was much greater for slabs in which the blow was cushioned than in slabs which received the force of the blow directly.

"The results of the tests of these specimens show that there was little if any gain in impact resistance due to the use of a large amount of reinforcing steel." This latter fact will seem reasonable in view of the fact that failures were by diagonal tension.

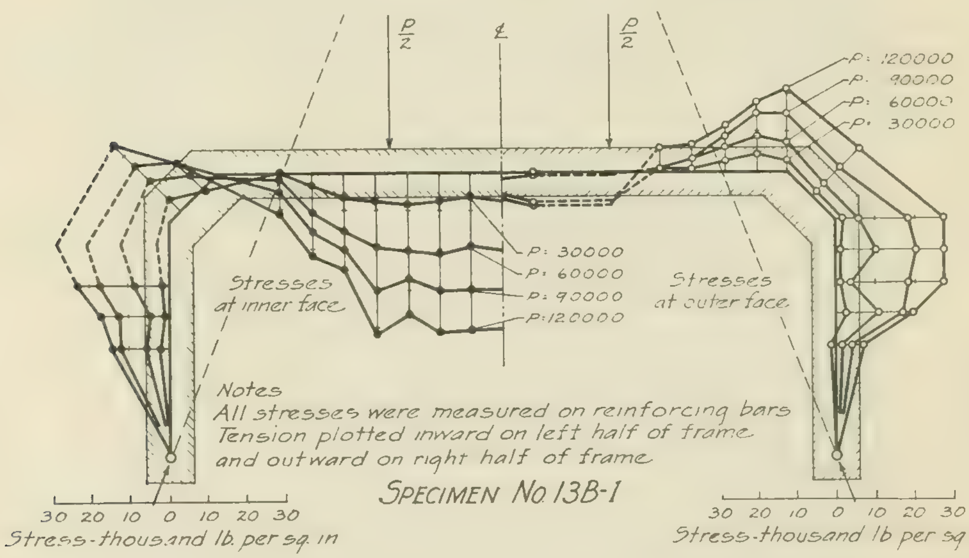


FIG. 19.—OBSERVED STRESSES IN FRAME 13B1.

"The use of "the cushion block reduced greatly the force of the blow for a given height of drop and at the same time caused the effect of the blow to be spread over a greater area. This resulted in an increase in the section of shear (diagonal tension) failure, and consequently, greater height of drop, and blows of greater force were necessary to cause failure. The use of the cushion block brought out to better advantage the effect of the thickness of slab, effect of amount of steel, and also the effect of span on impact resistance."

The force of the blow will be proportional to the negative acceleration developed in stopping the ball. Since the concrete slab is so much stiffer than the steel slab, the force exerted on the concrete slab would be much greater than that on the steel slab, even though the height of the fall were the same.

Although the steel shell resisted structural failure under impact much more effectively than the concrete slabs, it is reasonable to expect that the large detrusion which occurs with diagonal tension failure of the slab would

absorb the energy of the moving ball so rapidly that further resistance to impact would be more nearly equal for the concrete slab to that for the steel slab. No tests were made which bear exactly on this point.

EFFECT OF DIRECTION OF SLAB REINFORCEMENT ON STRESSES DEVELOPED.

In the design of reinforced concrete slabs it has generally been assumed that where the reinforcing bars make an angle with the direction of span the stress developed in the bar is equal to the product of the stress, which would be developed if the bars were in the direction of the span, and the secant of the angle which they make with the span. The writer knows of no tests which had been made to give information on this subject previous to those made by the Concrete Ship Section.

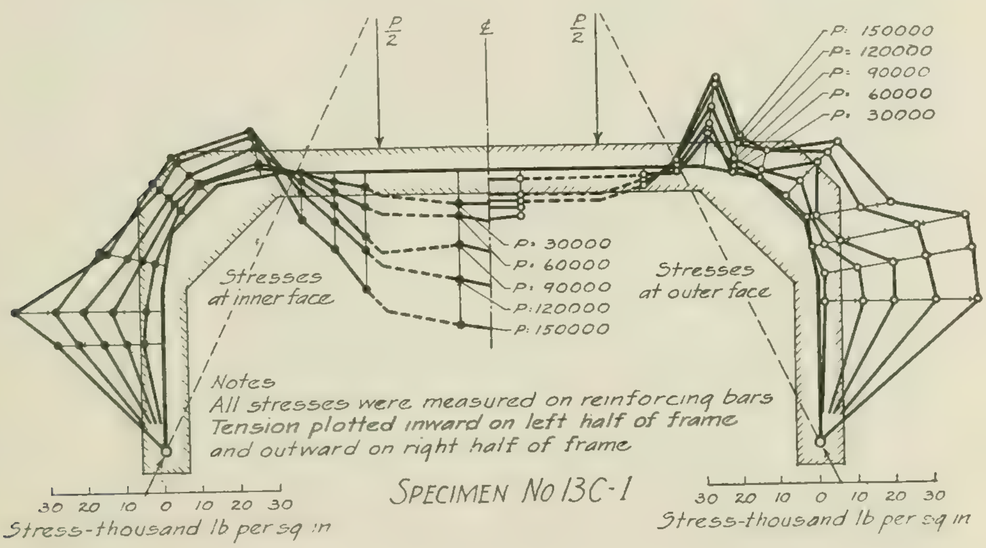


FIG. 20.—OBSERVED STRESSES IN FRAME 13C1.

Slabs were designed in which the reinforcing bars made an angle of 45° with the direction of the span, and for comparison with them slabs were made with the reinforcing bars placed in the direction of the span. The results indicated a stress in the slabs with bars at 45° about 20 to 25 per cent greater than in the case where the bars were parallel with the direction of span. With the number of tests available it seems best to interpret the results as meaning that the ordinary method of design is satisfactory though conservative, rather than that standards may be changed to permit greater advantage to the bars placed in the diagonal direction than is now accorded them.

In this investigation specimens reinforced with expanded metal were also included. These slabs show measured stresses in the reinforcement slightly less than the stresses in the slabs in which the same weight of steel was used in the form of bars placed in the direction of the span. Although this would

indicate that expanded metal is more effective in resisting tension than the same amount of metal would be if used in the form of bars extending in the direction of the stresses to be resisted, the tests have not been extensive enough to warrant so sweeping a conclusion. The showing is favorable to expanded metal, so far as economy of material is concerned, but more tests using deeper slabs should be made before this question is considered to be settled. The slabs used were so shallow that a small variation in depth would result in a relatively large variation in stress developed.

EFFECT OF BRACKETS ON BENDING MOMENT AT CENTER OF SPAN OF REINFORCED CONCRETE BEAMS

Regulations for design of beams or frames in which brackets, or haunches, are present are such as to prevent making any appreciable use of the brackets in reduction of bending moments at the center of the span of the bracketed

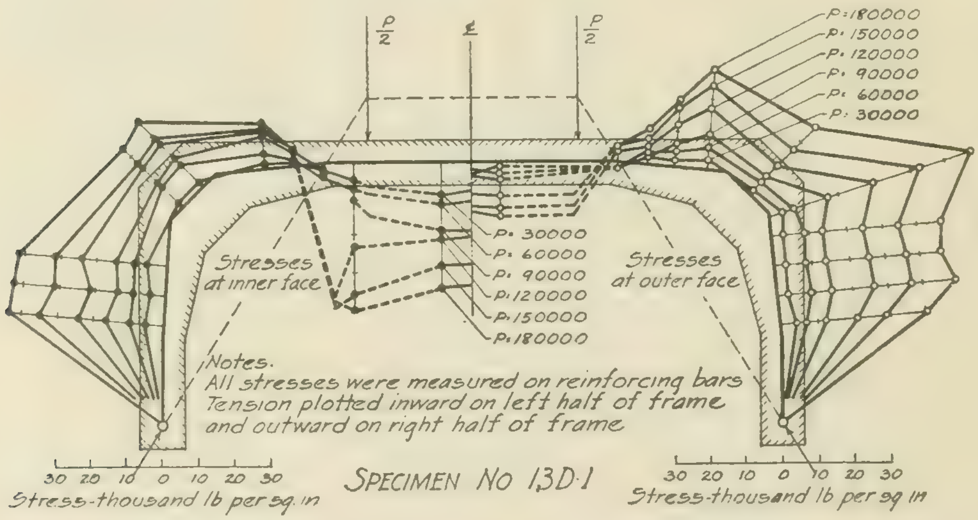


FIG. 21.—OBSERVED STRESSES IN FRAME 13D1.

beam. Analysis indicates that brackets are much more effective than is recognized in such regulations. In order to obtain information on the effectiveness of such brackets in modifying the distribution of bending moments in a frame, simple U-frames having a height of 7 ft. and a horizontal span of 14 ft. were tested. The frame was freely supported at the lower ends of the legs and load was applied at the one-third points of the span of the horizontal beam. Brackets of different sizes and shapes were used at the intersection of vertical and horizontal members in different specimens.

Figs. 18 to 21 give the stresses developed in the reinforcing bars at various points for frames of four different designs all having the same span length, height, and cross-section, except where the variation in size of brackets modified the section. The effect on the distribution of moments, due to variations in the size of brackets is apparent in these figures. The results of this series of tests indicate that more allowance for effect of brackets than is generally

recognized would be justified. The following statements are taken from the report of Mr. Richart who was in charge of this investigation:

"In so far as moment distribution is concerned, the entire section of a specimen may be considered effective, even at points of sudden change of shape, such as are seen in these specimens.

"The practice of using a bracket without determining its effect is not to be encouraged. The large effect of the bracket on moment distribution may cause a high stress to occur at a section where without the bracket it would not be expected and which is not reinforced to resist this stress.

"In conclusion, the result of these tests confirms the theoretical deduction that the use of brackets will result in a considerable saving of material and of

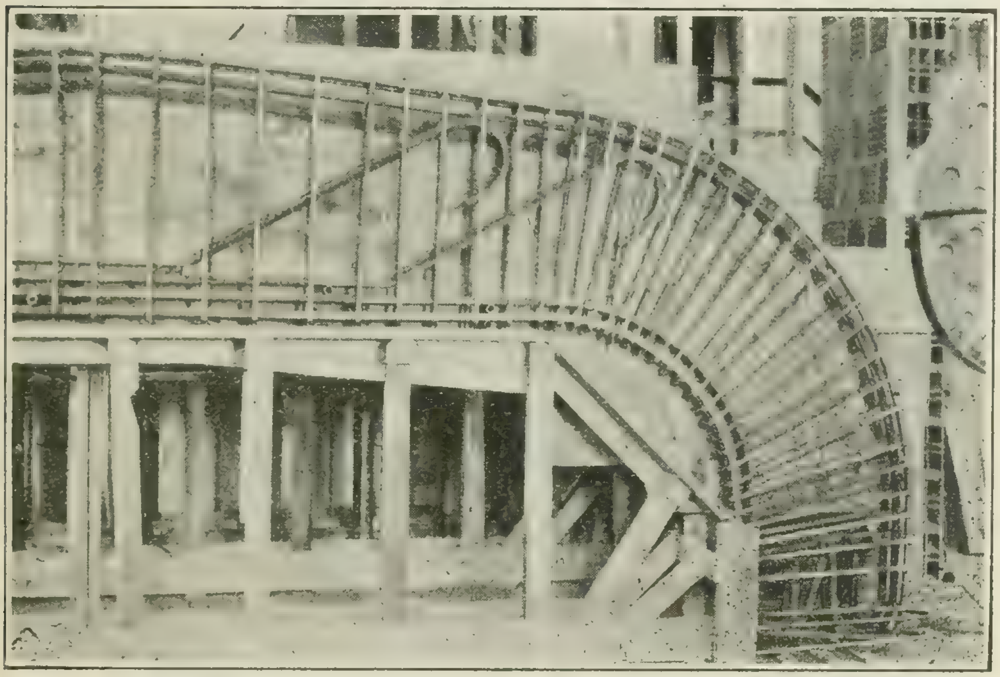


FIG. 22.—VIEW OF REINFORCEMENT USED IN SHIP FRAME SPECIMEN.

dead weight in a ship, and furthermore, that for a number of reasons their intelligent use produces desirable features in the structure."

TESTS OF SHIP FRAMES.

To verify the shear design of the frames of the 3500-ton concrete ship EF2, (of approximately the same design as the "Polias" built at the Fougner yard, Flushing Bay, L. I., and launched May 22, 1919), tests were made on frames built to full-sized cross-section and half length. The length of the frame out to out was 23 ft. 3 in., one-half the beam of the EF2, and the span between supports was 20 ft. 0 in. The depth of the main member was 3 ft. 8 in. at the center of the span, the same as the depth of the frames of the EF2. The frame was designed to carry a working stress in shear of 400 lb. per sq. in. and

at failure of the frames in the tests the shearing stress generally reached about 1500 lb. per sq. in. and failure was generally by tension in the longitudinal reinforcement. Fig. 22 shows the reinforcement of one of the typical test frames. Fig. 23 shows a test frame in the testing machine, and Figs. 24 and 25 show the appearance of opposite sides of one of the frames after the test. In Fig. 24 the cracks had been painted to make them visible in the photograph. In the test the bent-up longitudinal reinforcement shown in Fig. 22 carried larger stresses than the vertical stirrups. This, in connection with other

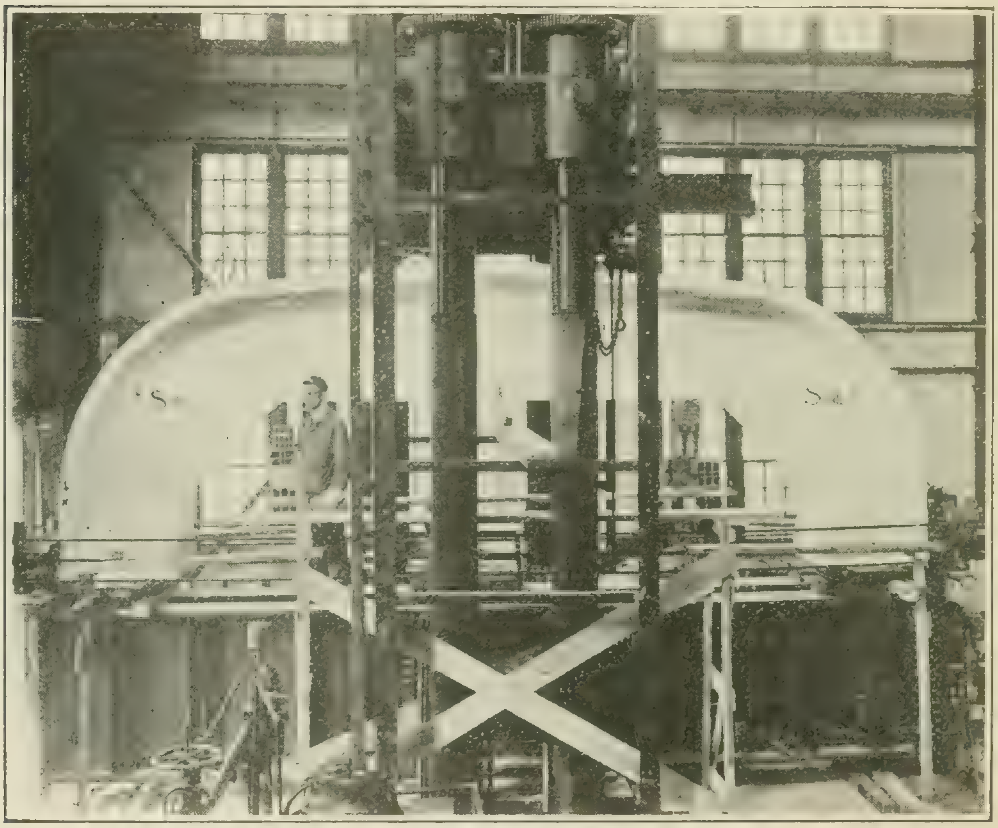


FIG. 23.—VIEW OF SHIP FRAME SPECIMENS IN 10,000,000-LB. TESTING MACHINE, BUREAU OF STANDARDS, PITTSBURGH.

data developed in the other portions of the shear investigation, indicates the value of bending up of longitudinal bars in addition to using vertical stirrups.

INVESTIGATION OF EFFECTIVENESS OF WEB REINFORCEMENT.

The frame tests described in the previous article were made to insure the safeness of the design for shear in the frames of the EF2 and no tests were made for comparison of various possible methods of reinforcement of the web. Likewise, beams were tested which had thin webs reinforced in a manner similar to that of the shell of the concrete ship, EF2. This was necessitated by the fact that the working stress in shear used in the design of the shell of

the concrete ship was much greater than is generally recognized as safe. However, at the same time that the tests were laid out for the purpose of checking the design, other tests were laid out to obtain information on other methods of web reinforcement. Later this led to the extensive investigation of some of the fundamentals of the action and value of web reinforcement. When the work had gone as far as it was believed was justified by the requirements for concrete ship design the shear investigation was extended under the auspices of the Bureau of Standards to obtain the information necessary for

FIG. 24.—VIEW OF SHIP FRAME AFTER TEST; CRACKS NOT PAINTED.

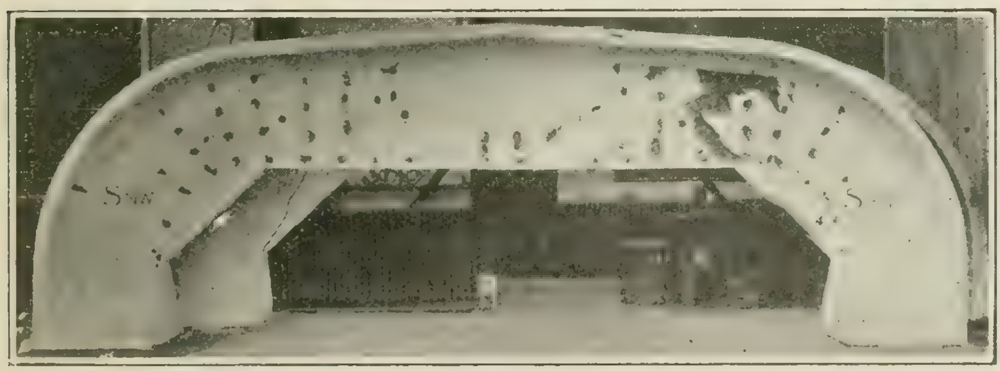


FIG. 25.—VIEW OF SHIP FRAME AFTER TEST; CRACKS PAINTED.

the application of the results to the more general field of reinforced-concrete design. In all nearly 200 beams have been tested.

It is not intended in this paper to give detailed results of the shear tests. Reference is made to the *Engineering News-Record* of Nov. 14, 1918, and Feb. 27, 1919, for certain results which have already been published. Attention is also called to a note in the issue of Mar. 20, 1919, correcting certain typographical errors which occurred in the article of Feb. 27.

Most of the beams were 10 ft. 8 in. long and 36 in. deep, 12 in. wide at flanges and of web thickness varying from 2 in. up to 12 in. The reinforcing bars were of steel rejected from shrapnel manufacture, and had a yield-

point stress of approximately 60,000 lb. per sq. in. The web reinforcement was of round bars generally spaced at distances of 4 in. in a direction normal to the axis of the bar. They were hooked over the longitudinal reinforcing bars at the top and at the bottom of the beam, but were not welded. Four different grades of concrete ranging in strength from 2000 to 5500 lb. per sq. in. were used.

The results of the tests are being studied, and it is expected that a synoptic report will be published by the Bureau of Standards at an early date, followed as soon as possible with a report giving the results as much in detail as the magnitude of the task and the number of the data will permit and justify. After the first few tests had indicated where measurements should be taken to obtain data of the most value the number of observations in each beam was reduced to a minimum, and it is believed that very few data have been taken which will not be of direct value in interpreting the results. Yet a report showing locations of all gage lines, the deformations, deflections, and crack widths which were measured, the manners of failure of the beams, and the make-up and strength of the concrete, together with ample discussion of the results would probably require more than 1000 pages. The writer believes this kind of a report would be justified by the value of the material, yet it does not appear at the present time that there will be financial support for such a program.

The study which has already been made has led to the development of an analysis which in a number of respects leads to conclusions which are confirmed qualitatively or quantitatively by the results of the tests. With the hope that its presentation may furnish a background for the consideration of some of the most general results, which are presented later, this analysis is given in the following pages

Analysis of Stresses in Inclined Web Reinforcement.—Imagine the web of a beam to be replaced by a double system of web members represented by EF and GH, as illustrated in Fig. 26, and the longitudinal tension and compression zones to be replaced by the horizontal members, BC and AD, respectively. Imagine all web members to be connected to the longitudinal members at E, F, G, H, etc., and to the posts, AB and CD, by perfectly hinged joints, but to be entirely free from any kind of articulation with each other at intersections. This structure is no longer a beam since stress is transferred from the upper to the lower chord as direct stress instead of shear, but it is sufficiently like some of the beams which have been tested in this investigation to present an analogy which will be useful in visualizing the action which takes place in these beams.

With the loads applied as shown, it is obvious that the following conditions will exist:

(*a*) A force, F_c, in compression will be applied to AD at D and a force, F_t, in tension will be applied to BC at C.

(*b*) $F_c = F_t = \dfrac{M}{jd}$.

(*c*) AD will shorten and BC will lengthen.

(*d*) The diagonal web members will assist in resisting the shortening of AD and the lengthening of BC.

(*e*) If all web members are equally capable of resisting a tendency towards axial deformation, whether in tension or compression, each web member will relieve the horizontal members of the same amount of tension or compression.

The total stress, $\frac{M}{jd}$, is removed from AD or BC in the distance l. Therefore the stress, ΔF, which is removed from AD or BC in the distance s is

(1) $\Delta F = \frac{Ms}{jdl}$, where

s = horizontal distance between web members
l = distance from load point to support.

Since in each distance s one diagonal tension and one diagonal compression member meet either longitudinal member, the horizontal component of the total stress in any one diagonal is $\frac{\Delta F}{2} = \frac{Ms}{2\ jdl}$ and the resultant total stress P_d, in the diagonal direction is

(2) $P_d = \frac{\Delta F}{2\ cos\theta} = \frac{Ms}{2\ jdl\ cos\theta}$, where

θ = angle of web members with horizontal.

It is desired to state this resultant stress in terms of the spacing a of web members measured at right angles to the direction of the members.

(3) $s = \frac{a}{sin\theta}$

Substituting this value of s in equation (2)

(4) $P_d = \frac{Ma}{2\ jdl\ cos\theta\ sin\theta}$

When $\theta = 45°$; $cos\theta\ sin\theta = 0.5$, and

(5) $P_d = \frac{Ma}{jdl}$

But $\frac{M}{l} = \frac{W}{2} = V$, whence

(6) $P_d = \frac{Va}{jd}$

Now assume that the web members entirely fill up the web of the structure, but that the members are still entirely distinct from each other and that both the tension members and the compression members can occupy the same space and still be free to act separately. Equation 6 may be written:

(7) $P_d = \frac{Va}{jd} = \frac{vbjda}{jd} = vba$, where

v = shearing unit stress and
b = thickness of web.

It is seen that this value of P_d is the same as the direct stress at 45° at the neutral axis in that portion of the web of a homogeneous beam which is included between two lines a very small distance a apart and making an angle of 45° with the horizontal. The structures under consideration are neither purely trusses, as was first assumed, nor are they homogeneous beams, but they must lie between these two cases. Since the same value for P_d is obtained for the two extreme cases, an analysis of an intermediate form of structure should give the same result, and the value obtained for P_d should represent the load carried by the stirrups in a reinforced-concrete beam with 45° tension web reinforcement. In such a beam the concrete must furnish the compression members. That the web stresses will act at approximately 45° is indicated by the direction of the tension cracks.

In equation (6) P_d represents the load carried by a tension stirrup.

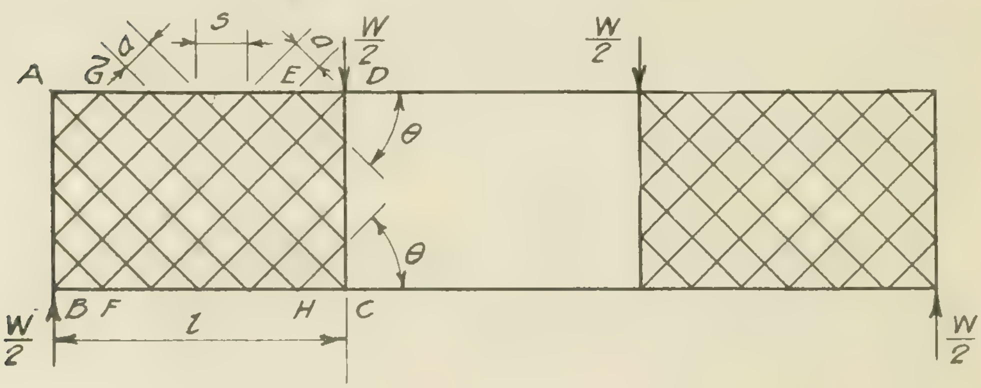


FIG. 26.—TRUSS-BEAM WITH DIAGONAL WEB MEMBERS.

This may be written:

(8) $P_d = vba = Af_s$ and

(9) $v = \frac{A}{ba} f_s = rf_s$

(10) $\frac{v}{f_s} = r$, in which

A = sectional area of a stirrup

f_s = tensile stress in stirrup

$r = \frac{A}{ba}$ = ratio of web reinforcement. ✓

Since for beams of the same depth and the same web thickness the ratio r is proportional to the quantity of steel used as web reinforcement, it has been found to afford a convenient basis for comparing various beams.

Analysis of Stresses in Vertical Web Reinforcement.—Assume now that the structure is made up of a system of diagonal compression members and with a vertical for each diagonal, as shown in Fig. 27, but with no diagonal

tension members. It is apparent that the verticals cannot take any of the horizontal component of stress from the longitudinal members and that in this case only one member (a diagonal) capable of taking a horizontal component of stress meets the members AD and BC in the distance s. Consequently, in this case the total stress carried in the diagonal member is twice as great as for the case in which diagonal tension members were present.

$$(11)\quad P_d=\frac{\Delta F}{\cos\theta}=\frac{Ms}{jdl\cos\theta}=\frac{Vs}{jd\cos\theta}$$

The diagonal members are in compression and each vertical member must resist as tension the vertical component of the compression in a diagonal. It is apparent that the vertical component P_v is

$$(12)\quad P_v=P_d\sin\theta=\frac{Vs\sin\theta}{jd\cos\theta}$$

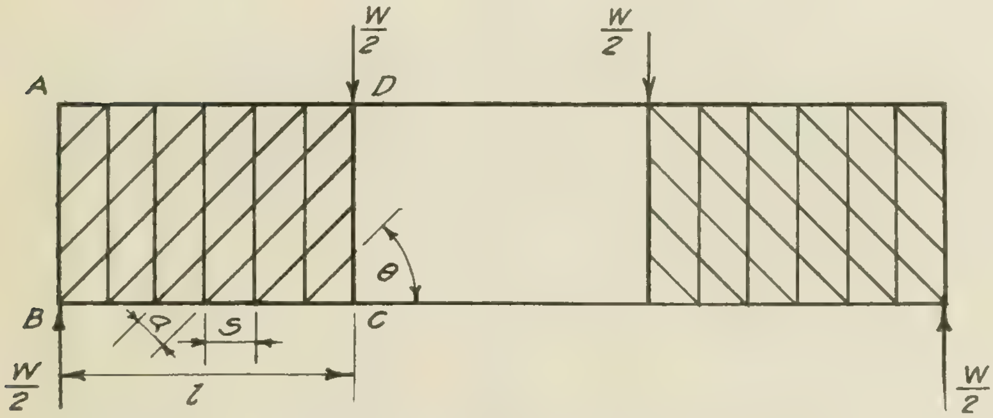


FIG. 27.—TRUSS-BEAM WITH VERTICAL AND DIAGONAL WEB MEMBERS.

When $\theta=45°\ \frac{\sin\theta}{\cos\theta}=1$ and

$$(13)\quad P_v=\frac{Vs}{jd}$$

$$(14)\quad \frac{Vs}{jd}=\frac{vbjds}{jd}=vbs=Af_s$$

$$(15)\quad v=\frac{A}{bs}f_s=rf_s$$

$$(16)\quad \frac{v}{f_s}=r$$

A comparison of equations (10) and (16) indicates that the tensile stress in the web reinforcement of beams with vertical bars is the same as that in beams with bars at 45° in the tension direction when the shearing unit stresses and the ratios of web reinforcement are the same.

It is quite generally assumed that the web reinforcement will be called on to resist only two-thirds of the web stresses. Applying this assumption in equation (13) there results the formula generally used in proportioning vertical stirrups:

$$(17)\quad P = \tfrac{2}{3}\,\frac{Vs}{jd}$$

From this

$$(18)\quad v = 1.5\, rf_s$$

$$(19)\quad \frac{v}{f_s} = 1.5\, r$$

Analysis of Diagonal Compressive Stresses.—By the use of equation (7) the diagonal compressive unit stress in the concrete for beams with diagonal tension reinforcement may be written:

$$(20)\quad f_c = \frac{P_d}{a\,b} = \frac{Va}{abjd} = \frac{V}{bjd} = v$$

This indicates that for this case the diagonal unit compressive stress is equal to the vertical unit shear at the neutral axis. From equation (11) the diagonal compressive unit stress in the concrete for beams with vertical reinforcement is:

$$(21)\quad f_c = \frac{P_d}{ba} = \frac{Vs}{bjd\, s \sin\theta \cos\theta} = \frac{v}{\sin\theta \cos\theta}$$

When $\theta = 45°$ $\sin\theta \cos\theta = 0.5$, and

$$(22)\quad f_c = 2\,\frac{V}{bjd} = 2v$$

This indicates that the diagonal compressive stress in a beam with vertical web reinforcement is twice as great as the diagonal compressive stress in a beam with diagonal tension web reinforcement.

Observed Tensile Stress in Vertical and Diagonal Reinforcement.—In the earliest part of the investigation it was observed that when the ratio of web reinforcement was the same for beams with vertical and for beams with diagonal reinforcement the tensile stresses developed in the reinforcement were equal. This confirms the conclusions arrived at by comparison of equations (10) and (16).

As to the amount of stress developed, the agreement between the test results and the analysis is not so good. For the purpose of this comparison, but without making recommendations of standards for design, it may be stated that independently of the strength of the concrete and the thickness of the webs the test results for loads causing tensile stresses in the web reinforcement of 40,000 lb. per sq. in. or more are represented with considerable accuracy by the equation:

$$(23)\quad \frac{v}{f_s} = 0.006 + 1.125\, r$$

The graph of this equation is shown in Fig. 28 for comparison with the graphs of equations (10) or (16) and (23).

It must be recognized that in equation (23) no correction has been made for the strength added by the frame action of the heavy flanges and pilasters. When this correction has been made it may be necessary to modify somewhat the equation representing the results. However, the investigation included some rectangular beams and the stresses developed in these beams seem to satisfy the equation about as well as those of the others.

As the results stand, it will be seen that the tensile stresses in the test beams were considerably lower than the results of analysis indicate might be

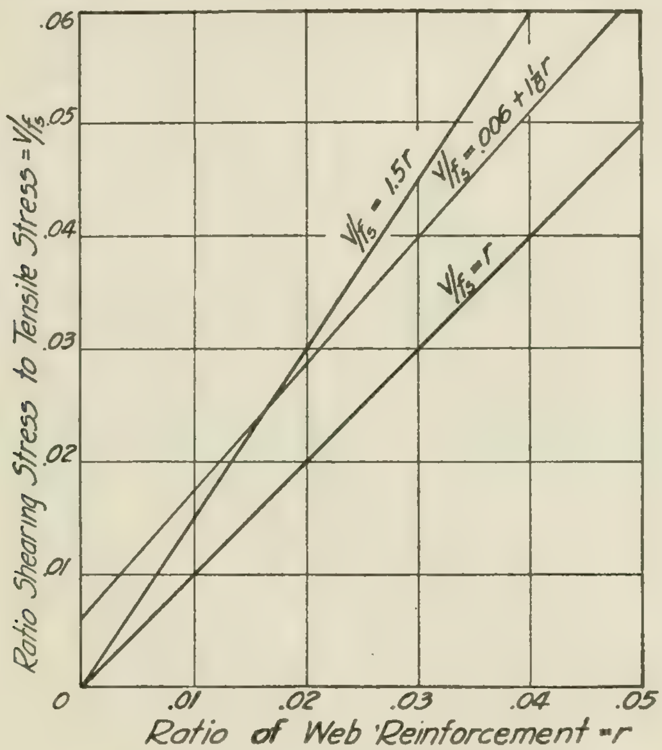


FIG. 28.—GRAPHS OF EQUATIONS FOR RELATIONS BETWEEN SHEAR, TENSILE STRESS AND RATIO OF WEB REINFORCEMENT.

expected. For less than 1.6 per cent of web reinforcement the tensile stresses developed in the tests were less than those given by equation (17) (the Joint Committee standard) and for larger amounts of web reinforcement the tensile stresses in the tests were greater than those given by equation (17). The maximum allowable shearing stress recognized by the Joint Committee standard, however, is 6 per cent of the compressive strength of the concrete, and this limits the web reinforcement to not more than 0.5 per cent for concrete having a compressive strength of 2000 lb. per sq. in. Hence, within the probable range of concrete strengths likely to be met in practice the Joint Committee recommendations are well within the limits indicated by the tests as permissible.

The tests indicate that by using larger amounts of web reinforcement than are now recognized, higher shearing stresses might safely be permitted,

but for the larger amounts of web reinforcement less allowance should be made for the reinforcement than is indicated in equation (17) the one now in almost universal use for design of web reinforcement.

Observed Diagonal Compressive Stress in Beams Tested.—In Fig. 29 are plotted from common origins for comparison the diagonal compressive unit deformation in beams with vertical and with diagonal web reinforcement for four different percentages of reinforcement. This diagram indicates that generally the compression was larger for the beams with vertical stirrups than for beams with diagonal stirrups. According to equations (20) and (22) it would be twice as great, but it does not seem from the tests to be as great as this. Each curve represents the results for only one beam and the results

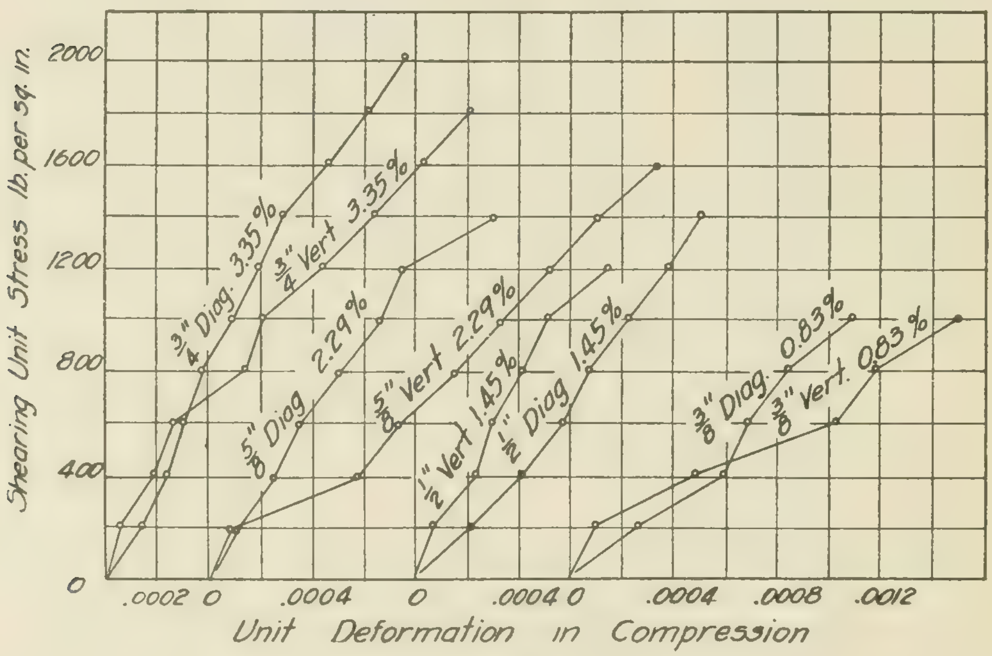


FIG. 29.—DIAGONAL COMPRESSION DEFORMATION FOR VERTICALLY AND FOR DIAGONALLY REINFORCED BEAMS

cannot be taken as conclusive. Other data are available for a fuller study of this subject.

Equation (22) indicates that the diagonal compressive unit stress in a beam with vertical stirrups is equal to twice the vertical unit shear. For the vertically reinforced beams which failed in diagonal compression the vertical shearing unit stress at failure was generally about one-half the compressive strength of the control cylinders. To this extent the results confirm the showing of equation (22). The same line of reasoning would require that the diagonally reinforced beams which failed in diagonal compression should develop a shearing stress equal to the compressive strength of the concrete

or twice that of the vertically reinforced beams. They did not do this, though they did develop higher shearing stresses before failing in diagonal compression than did vertically reinforced beams having concrete of the same strength.

Proper attention to keeping the compressive stresses within limits will prevent making the webs of beams too thin if there are no other considerations which bring this about. The test results would indicate that for beams with vertical stirrups the web reinforcement should not exceed more than about 1.7 per cent in order to maintain the usual relations between the tensile and the compressive stresses for concrete having a strength of 2000 lb. per sq. in.

With diagonal stirrups the reinforcement could apparently be somewhat larger.

Limitations to Applicability of Test Results.—The applicability of results of this investigation is somewhat limited by the following conditions which surround the tests:

(*a*) Most of the beams were rather deep relatively to the span.

(*b*) Most of the beams had thin webs and heavy frames, and the loads were probably increased somewhat by the action of the frames independently of the flexure of the beam as a whole.

(*c*) The spacing of the stirrups generally was small relatively to the depth of the beam.

So far as information on the significance of these limitations has been obtained, the indications are that the effect of the spacing of the stirrups is of much more importance than that of the other two limitations mentioned.

DISCUSSION.

Mr. Godfrey

Mr. Edward Godfrey (*by letter*).—Assuming that Mr. Slater's paper is in substance the same as the article in *Engineering News-Record*, Feb. 27, 1919, the writer wishes to discuss the paper and to point out that the apparent high unit stresses in shear found by Mr. Slater need scaling down.

In about 1907 a series of tests were published that seemed to show that concrete is capable of withstanding a unit shearing stress approaching its compressive strength. At that time the writer pointed out the fact that the tensile strength of concrete is so closely associated with the shearing strength that it was only in tests where tensile stresses in concrete were inhibited by steel reinforcement that high shearing unit stresses were found. Accordingly the showing of those tests was misleading if interpreted to give warrant for high unit shear, for the reason that in actual construction of beams the condition of those tests are not duplicated. In the tests referred to great pains seemed to have been taken to avoid tensile stress in the concrete by means that were not usual and were scarcely possible in construction. The tests made by Mr. Slater, in the writer's judgment, are also misleading, as interpreted by him, for two reasons. First, the beams are of very unusual shape for reinforced concrete, and second, the shear which Mr. Slater attributes to the concrete web is in reality divided between the concrete web and the heads or flanges of the beam; in addition there is the assistance rendered, after initial failure of the concrete, by the truss action of the steel reinforcement.

When the thing tested is unlike usual construction, the results obtained must be interpreted accordingly; and when a stress is taken simultaneously through several channels, it is not correct to attribute the entire work to one of those channels.

Mr. Slater, in his published tests, attributes the shear carrying capacity of the beams to the concrete web alone. This is not correct, for there are large heads or flanges on the beams which in themselves, unaided, are capable of carrying a very large part of the shear. This is proven by test *4AD*, which had no concrete web whatever and yet took a shear about half as great as a similar beam having a concrete web of nearly 90 sq. in. On exactly the same basis as Mr. Slater's other tests, by his method of figuring, this beam should show a unit shear of infinity.

The shear in these tests was in reality borne by the full section of the concrete, and the same was helped by the steel reinforcement (after the concrete had cracked), which in itself was quite heavy for the beams. These concrete heads were quite chunky, and not thin flanges like those of a plate-girder, to which Mr. Slater compares them. Mr. Slater has con-

tended that the flanges of a steel plate-girder are not counted in as resisting shear in a plate-girder design. The writer cannot see what the conventional methods of designing a plate-girder have to do with interpretation of results of tests on reinforced concrete. Some years ago proposals were seriously made to design steel girders with rectangular frames and no webs whatever. These proposals were backed by tests, so that a steel girder, too, can take considerable shear without a web. The flanges of a steel girder, of course, carry some shear, but the proportion carried would be different with every different shape of girder and flange. Hence it would be quite misleading to test a series of girders of a given shape and on the results of these tests to assert that girder webs are capable of carrying whatever unit shear happened to work out in the nominal web area of these girders. **Mr. Godfrey**

MR. W. A. SLATER (*by letter*).—Mr. Godfrey begins his discussion with the assumption that this paper is "in substance the same as the article in *Engineering News-Record*, Feb. 27, 1919." This assumption is incorrect, but it makes little difference in the result, for he seems to know little more of the contents of the *News-Record* article than of this paper, which he appears not to have seen. **Mr. Slater**

Although Mr. Godfrey's discussion does not bear on the present paper, there are two reasons for replying to it: (1) It contains misrepresentations of fact, which, whether it be intentional or not, should not pass unnoticed. (2) Each unanswered discussion Mr. Godfrey appears to regard as unanswerable and to make use of it in attempting to throw unnecessary confusion around the subject of web reinforcement.

The discussion states that "Mr. Slater in his published tests attributes the shear-carrying capacity of the beams to the concrete web alone." This statement is entirely untrue, and if Mr. Godfrey does not know it, it is because he has not read the *Engineering News-Record* article to which he refers or the author's reply* to the same statement previously made by Mr. Godfrey. In the original article† definite statements on this subject are made in two places, and again in the author's* reply to Mr. Godfrey's discussion. These are as follows: (a) "From the phenomena of the tests it is apparent that an appreciable portion of the load was carried directly by the frame action of the heavy pilasters and flanges which were built monolithically with the beam," and (b) "With the monolithic construction of the beam the flanges and pilasters would act as a frame to some extent independently of the beam structure, and would carry loads which did not have to pass through the web in shear. The amount of this error is not known, but it seems that it cannot be more than the total load carried by a beam like *4AG1* and *4AG2*, and it probably is somewhat less."

Even if Mr. Godfrey, on his own initiative, failed to read these statements, his attention was called to them in the following language,‡ in reply to his previous discussion: "Mr. Godfrey would have found by

* *Engineering News-Record*, April 17, 1919, p. 783

† *Engineering News-Record*, Feb. 27, 1919. p 430-433.

‡ *Engineering News-Record*, April 17, 1919, p. 784.

Mr. Slater. reading the article that I recognized that the shearing stress given 'must be somewhat in error, etc.' The only difference of opinion is as to how large this error is." In view of these circumstances, it is difficult to see how the statement that "Mr. Slater in his published tests attributes the shear-carrying capacity of the beams to the concrete web alone," can be other than a deliberate misrepresentation.

Mr. Godfrey notes that beam *4AD* "took a shear about half as great as a similar beam having a concrete web of nearly 90 sq. in." This, he seems to think, proves that the flanges "in themselves, unaided, are capable of carrying a very large part of the shear," but he fails to point out that this beam had reinforcement in the web which assisted greatly in carrying the load. If he would refer to beams *4AG1* and *4AG2*, which had neither concrete nor steel in the web, he would find the load carried to be very much smaller.

Mr. Godfrey claims as far back as 1907 to have "pointed out the fact that . . . . it was only in tests where tensile stresses in concrete were inhibited by steel reinforcement that high shearing unit stresses were found." Now that Mr. Godfrey is compelled to recognize that stirrups are effective, it is amusing to note that he claims to have been the discoverer of this fact in 1907. If in the past he has recognized the value of stirrups, what did he mean by his statement,* "The Joint Committee report does not state whether the stress in a shear member is shear, or compression, or tension. All three are equally absurd." The author has previously† asked for an explanation of this statement, which is similar in purport to many others made by Mr. Godfrey in recent years. Until such an explanation is forthcoming, what Mr. Godfrey pointed out in 1907 is of little consequence.

In a later paragraph Mr. Godfrey naïvely remarks that the shear was "helped by the steel reinforcement." Most engineers have known this for a long time. That is why the steel was placed in the beam. Mr. Godfrey alone has maintained that web reinforcement is of no use. Until he has reconciled his present statements, that the web reinforcement assists greatly in carrying the loads, with his many sweeping assertions as to the uselessness of web reinforcement, his criticisms need not be taken very seriously.

* Jour. A. C. I., Dec., 1914, p. 19.

† *Engineering News-Record*, April 17, 1919, p. 784.

EFFECT OF VIBRATION, JIGGING AND PRESSURE ON FRESH CONCRETE.

By Duff A. Abrams.*

Introduction.

An experimental study of the effect of vibration and pressure on fresh concrete on its strength and other properties is of interest in view of the frequent use of such devices as hand-hammering of forms, or air-hammering, jigging or vibration as an aid in placing concrete. Such methods are particularly applicable to the construction of reinforced-concrete ships and houses, where thin sections and a multiplicity of reinforcing members are of common occurrence. Jigging or vibrating machines are frequently used in concrete products plants. The effect of pressure on fresh concrete is of interest in certain problems of concrete design.

Little attention has heretofore been given to the experimental study of the effects produced by vibration and jigging fresh concrete. A few tests were made in a study of the effect of pressure on fresh cement paste in a confined space by James E. Howard, at Watertown Arsenal.[1] The effect of pressure on the compressive strength and bond was studied by the writer at the University of Illinois in 1913.[2] Since the tests reported herein were completed, Prof. F. P. McKibben has published a report on compression tests of concrete columns which set under pressure.[3]

The tests included in this report were made as a part of the experimental studies of concrete and concrete materials being carried out through the cooperation of Lewis Institute and the Portland Cement Association.

Outline of Tests.

The tests included in this report cover the following topics:

1. Different methods of hand-molding of test cylinders.
   (*a*) Puddling with ⅝-in. round steel bar (varying number of strokes).
   (*b*) Tamping (tampers of different size).
   (*c*) Tapping metal forms after puddling.
2. Effect of vibrating fresh concrete (small electric motor, Fig. 1.).
   (*a*) Time of vibration varied up to 1 min.
3. Effect of jigging fresh concrete (using machine shown in Fig. 2).
   (*a*) Concrete of different mixes (1:7 to 1:3).
   (*b*) Concrete of different consistencies (0.70 to 1.25).

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* Professor in Charge, Structural Materials Research Laboratory, Lewis Institute, Chicago.

[1] Cement Age, June, 1905.

[2] "Tests of Bond between Concrete and Steel." Bulletin 71, Illinois Engineering Experiment Station, 1914.

[3] Eng. News-Rec., Dec. 5, 1918.

(c) Using aggregate of different grading (fineness modulus 4.00 to 6.50).
(d) Using aggregate of different sizes (0-28 sand to 0-1½-in. concrete aggregate).
(e) Using coarse aggregate of different shape (pebbles and crushed stone).
(f) Effect of rate of jigging (0 to 150 r. p. m.).
(g) Effect of height of drop (0 to 0.50 in.).
(h) Effect of length of time jigged (up to 3 min.).
(i) Effect of age of concrete before jigging (up to 6 hr.).
(j) Jigged with 30-lb. weight on top of fresh concrete.
(k) Hand puddling on jigging machine while in operation.

4. Effect of pressure on fresh concrete (method of applying the higher pressures shown in Fig. 3).
(a) Using different pressures (0 to 500 lb. per sq. in.).
(b) Effect of duration of pressure (15 min. to 16 hr.).
(c) Effect of removal of water by pressure.

This series included 900 compression tests of 6 x 12-in. concrete cylinders at the age of 28 days. All specimens were made from the same materials at the same time, consequently direct comparisons may be made between any two sets of tests.

MATERIALS.

The portland cement used consisted of a mixture of equal parts of four brands purchased from Chicago dealers. The cement conformed to all the requirements of the Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials.

In general the aggregates consisted of sand and pebbles from the Chicago Gravel Company's pit near Elgin, Ill. In one group of tests crushed limestone was used as coarse aggregate. Sieve analysis and miscellaneous tests of aggregates are given in Table 1.

The "fineness modulus" of an aggregate may be used as a measure of its size and grading. It is the sum of the percentages in the sieve analysis divided by 100. The sieve analysis is expressed in terms of weight or volume *coarser* than each sieve. Tyler standard screen scale sieves are used. The sizes of sieves and fineness modulus of the aggregates used in these tests are shown in Table 1. Low values of fineness modulus (abbreviated to F. M. in the tables) correspond to small size and high values to the coarse sizes of aggregate.*

TEST PIECES.

All test pieces consisted of 6 x 12-in. cylinders which were stored in damp sand for 28 days. The concrete for each specimen was proportioned separately and mixed by hand with a bricklayer's trowel in a shallow metal pan. The

* For further details on fineness modulus of aggregates see the writer's report on "Design of Concrete Mixtures," Bulletin 1, Structural Materials Research Laboratory.

forms consisted of 12-in. lengths of cold-drawn steel tubing, split along one element. Each form stood on a machined cast-iron base plate. A smooth top was formed by means of neat cement and plate glass.

Unless otherwise noted, the specimens were molded by the "standard" hand-puddling method before subjecting them to vibration, jigging or pressure. This method consists of puddling the fresh concrete in the metal form in 4-in. layers by means of 25 strokes with a ⅝-in. round steel bar and leveling off with a trowel. This method has been in use for several years in our research work and has been found to give uniform results for different operators. The strength of the concrete produced by this method of molding is used as a basis for comparison (100 per cent.) for all other methods of treatment included in this investigation.

TABLE 1.—MISCELLANEOUS TESTS OF AGGREGATE.

Size	Kind.	Weight lb. per cu. ft.	Density.	Sieve Analysis of Aggregate. (Per cent by weight coarser than each sieve.)									Fineness Modulus.*
				100	48	28	14	8	4	⅜ in.	¾ in.	1½ in.	
0–28	Elgin Sand and Pebbles	102	0.61	88	42	0	..	..	..	..	..	..	1.20
0–14		104	0.62	90	91	36	0	..	..	..	..	..	2.25
0–8		108	0.65	99	93	50	22	0	..	..	..	..	2.64
0–4		112	0.67	99	94	59	36	18	0	..	..	..	3.06
0–⅜ in.		120	0.72	99	96	73	57	46	33	0	..	..	4.04
0–¾ in.		124	0.74	99	97	82	72	64	58	30	0	..	5.02
0–1½ in.		128	0.77	99	98	87	80	75	68	48	20	0	5.75
0–1½ in.	Elgin Sand and Pebbles	119	0.71	99	95	69	51	38	24	17	7	0	4.00
		125	0.75	99	97	79	68	58	49	35	15	0	5.00
		126	0.76	99	98	85	76	69	62	43	18	0	5.50
		128	0.77	99	98	87	80	75	68	48	20	0	5.75
		128	0.77	100	98	90	84	79	75	52	22	0	6.00
		125	0.75	100	99	92	88	84	81	57	24	0	6.25
		120	0.72	100	99	95	92	90	87	62	25	0	6.50
0–1½ in.	Elgin Sand and Crushed Limestone	123	0.74	99	95	69	51	38	24	17	7	0	4.00
		128	0.77	99	97	79	68	58	49	35	15	0	5.00
		124	0.74	99	98	85	76	69	62	43	18	0	5.50
		122	0.73	99	98	87	80	75	68	48	20	0	5.75
		118	0.71	100	98	90	84	79	75	52	22	0	6.00
		114	0.68	100	99	92	88	84	81	57	24	0	6.25
		110	0.66	100	99	95	92	90	87	62	25	0	6.50

* The sum of percentages in sieve analysis, divided by 100.

The mixture is expressed as one volume of cement to a given number of volumes of mixed aggregate. A 1:5 mix expressed in this manner is about the same as the ordinary 1:2:4 mix. The exact equivalent of the latter will vary with the size and grading of the aggregates.

The water content of the concrete is expressed in terms of the relative consistency and the "water-ratio." A relative consistency of 1.00 (normal consistency) is of such plasticity that the concrete of usual mixes will slump ½ to 1 in. if the metal form is withdrawn by a steady upward pull immediately after molding the cylinder by the standard method. A relative consistency of 1.10 contains 10 per cent more water than normal consistency. The water-ratio is the ratio of volume of water to volume of cement in the batch. The

weight of cement was assumed as 94 lb. per cu. ft. For one mix and given concrete materials the relative consistency and water-ratio may be used interchangeably.

In the hand-molded specimens the method of placing the concrete was varied by changing the number of strokes of the puddling bar, using layers of different thickness, etc. Hand-tampers 2 in. and 5 in. in diameter were also

FIG. 1.—ELECTRIC VIBRATOR.

Shows set-up for vibration tests given in Table 3.
Motor weighed 12 lb., ran about 1000 r. p. m.

used. In one set of tests the form was struck with a steel bar after molding by the standard puddling method.

In the vibration tests the cylinder mold was bolted to a light timber table and the concrete specimen molded by the standard hand-puddling method described above.

Violent vibration was produced by holding an electric motor frame against the side of the steel form as shown in Fig. 1. The motor carried an

eccentric flywheel, weighed 12 lb. and ran about 1000 r. p. m. The time of vibration varied from 5 sec. to 1 min.

The jigging tests were made on the machine shown in Fig. 2. The machine consisted of a framework carrying a metal table about 4 ft. wide and 8 ft. long weighing about 700 lb. The table was raised by means of belt-driven cams on two longitudinal shafts. The rate and height of drop could be varied over a wide range. In most of the tests the machine was run for 20 sec. at 100 drops per min., 0.1-in. drop; however, each of these factors were varied with the other two constant.

All specimens were molded by the standard hand-puddling method before pressure was applied. Pressures up to 10 lb. per sq. in. were applied by piling weights on top of a loose-fitting cover plate. Pressures of 25 and 50 lb. per

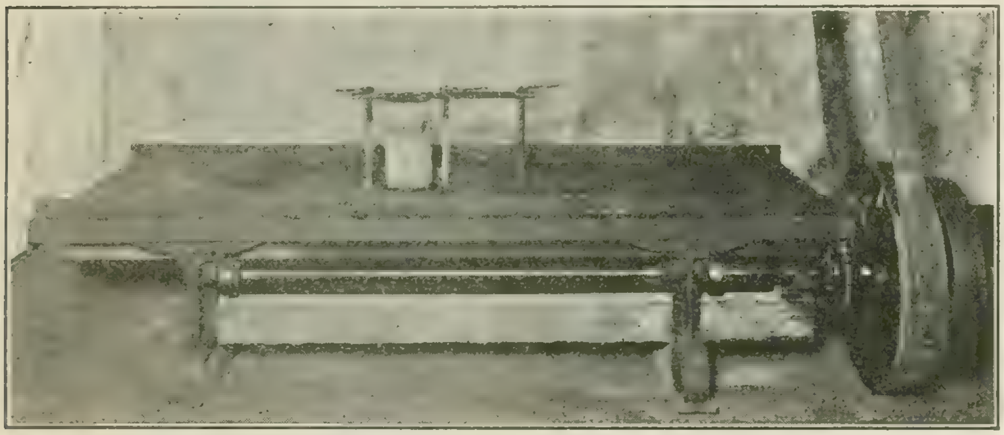


FIG. 2.—JIGGING MACHINE.

Steel table 4 x 8 ft.; weight 700 lb. Table raised by means of cams; rate and height of drop can be varied over wide range.

sq. in. were applied by weighted levers. Pressures of 100 to 500 lb. per sq. in. were obtained by placing the freshly molded specimen in a testing machine, as shown in Fig. 3. The spring facilitated maintaining a constant pressure. For all pressures the time of application varied from 15 min. to 16 hr.

The metal forms are fairly tight and permit little leakage of water under ordinary conditions. For the wetter concretes the joints were sealed with paraffin. In the pressure tests the water expelled was collected by means of sponges and weighed. It should be noted that this water was almost clear.

Test cylinders were made in sets of 5 on different days. One specimen of each form was made before starting the second round. The strengths given in the tables are the average of 5 entirely independent tests. In this way minor variations in materials, proportions, manipulation, etc., are eliminated. In the case of the "standard" hand-puddled specimens which are used as a basis of comparison, 3 sets of 5 specimens were made in different parts of the series.

TEST DATA AND DISCUSSION.

Data of the tests will be found in Tables 2 to 7. Only average values are reported. It will be noted that the "standard" hand-puddled concrete (average of 15 tests) is used as the basis of comparison. The diagrams in Figs. 4 to 16 give the data in graphical form.

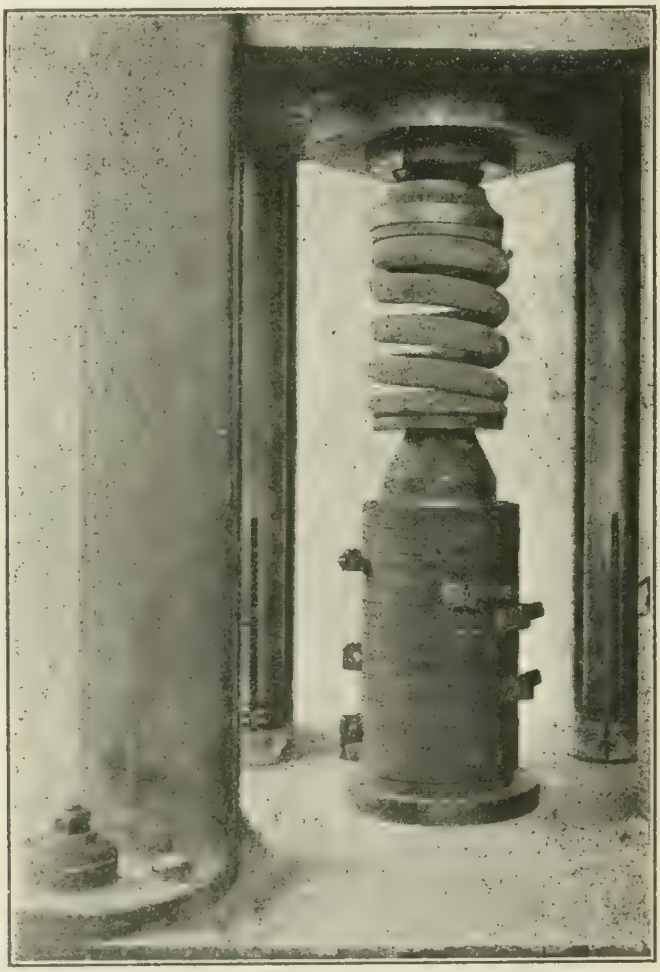


FIG. 3.—METHOD OF APPLYING PRESSURE TO FRESH CONCRETE.

The higher pressures were applied by means of a testing machine. The spring facilitated maintaining a constant pressure. The plate in contact with fresh concrete was loose-fitting. The water expelled from concrete was collected by means of sponges and weighed.

A comparison of the relative effects of puddling 1:5 plastic concrete with a steel bar and tamping with tampers of different weight and size is given in Table 2.

The effect of vibration with electric vibrator is shown in Table 3 and Fig. 4. Vibrating for about 30 seconds caused no appreciable effect on the strength of concrete which had been puddled in place by hand. The tests

TABLE 2.—EFFECT OF METHOD OF MOLDING CONCRETE SPECIMENS.

Compression tests of 6 x 12-in. cylinders.
Age at test 28 days; stored in damp sand; tested damp.
Aggregate—Sand and pebbles from Elgin, Ill., graded 0–1½ in.
Each value is the average of 5 tests made on different days.

Ref. No.	Mix by Volume.	F. M. of Aggregate.	Relative Consistency.	Water-Ratio to Volume of Cement.	Compressive Strength lb. per sq. in.	Compressive Strength Per cent of Standard.	Treatment of Concrete
1	1:5	5.75	1.00	0.875	2680	96	12 strokes around perimeter of form for each 4-in. layer of concrete using ⅝-in. steel bar.
31	....	....	....	....	2800		25 strokes distributed over section for each 4-in. layer using ⅝-in. steel bar. (Standard method).
51	....	....	....	....	2710		
84	....	....	....	....	2840	100	
					2780*		
2	....	....	....	....	2810	101	50 strokes distributed over section for each 4-in. laye using ⅝-in. bar.
147	....	....	....	....	2690	97	12 strokes on each 3-in. layer using 2-lb. tamper 2 in. in diameter.
148	....	....	....	....	2420	87	12 strokes on each 6-in. layer, using 2-lb. 2-in. tamper.
149	....	....	....	....	2430	87	12 strokes on first 3-in. layer using 2-lb. 2-in. tamper, forms then filled before tamping again with 12 strokes.
150	....	....	....	....	2500	90	12 strokes on first 3-in. layer, using 2-lb. 2-in. tamper, remaining concrete settled by tapping form lightly.
3	....	....	....	....	2800	101	25 strokes distributed over section for each 4-in. layer with 2-lb. 2-in. tamper.
4	....	....	....	....	2570	92	25 strokes distributed over section for each 4-in. layer with 2-lb. 5-in. tamper.
5	....	....	....	....	2740	98	Standard method of molding, except form struck 3 light blows with steel bar after puddling each 4-in. layer.

* Average of 15 tests made on different days. This value is used as a basis for comparison in Tables 2, 3, 6 and 7.

TABLE 3.—EFFECT OF VIBRATION WITH ELECTRIC MOTOR.

Method of vibrating shown in Fig. 1.
Compression tests of 6 x 12-in. cylinders.
Results of tests are platted in Fig. 4.
Age at test 28 days; stored in damp sand; tested damp.
Aggregate—Sand and pebbles from Elgin, Ill., graded 0-1½ in
Each value is the average of 5 tests made on different days.

Ref. No.	Mix by Volume.	F. M. of Aggregate.	Relative Consistency.	Water-Ratio to Volume of Cement.	Compressive Strength		Treatment of Concrete.
					lb. per sq. in.	Per cent of Standard.	
31, 51, 84	1:5	5.75	1.00	0.875	2780*	100	Standard method of molding.
6	....	....	....	....	2880	104	Standard method of molding. Vibrated 5 sec.
7	....	....	....	....	2640	95	Standard method of molding. Vibrated 10 sec.
8	....	....	....	....	2830	102	Standard method of molding. Vibrated 20 sec.
9	....	....	....	....	2700	97	Standard method of molding. Vibrated 30 sec.
10	....	....	....	....	2520	91	Standard method of molding. Vibrated 45 sec.
11	....	....	....	....	2470	89	Standard method of molding. Vibrated 60 sec.

* Average of 15 tests made on different days, See Table 2.

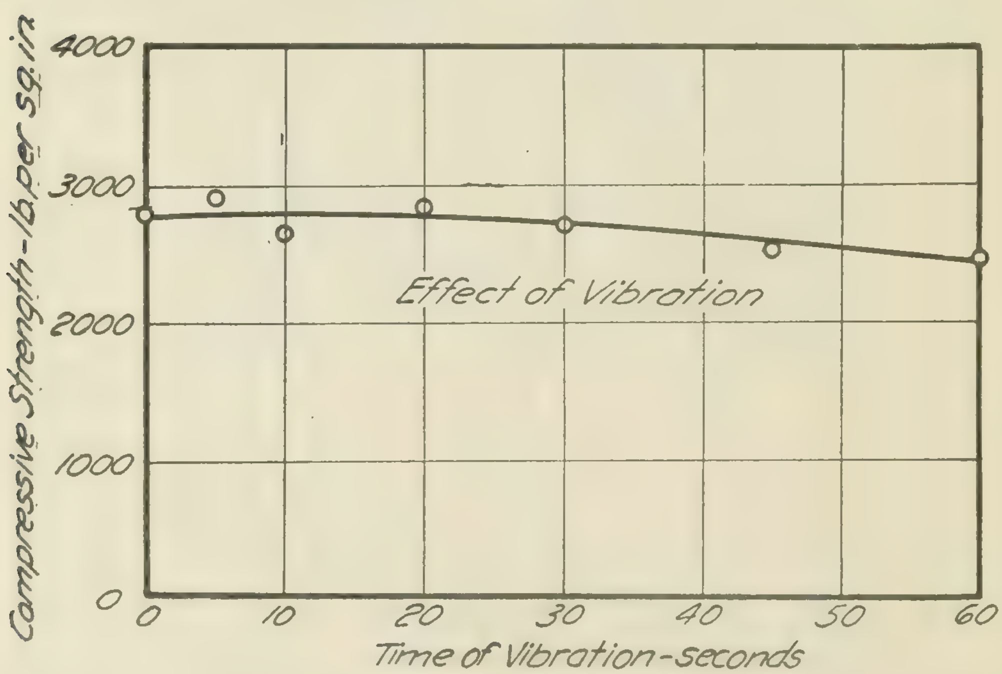


FIG. 4.—EFFECT OF VIBRATION ON THE STRENGTH OF CONCRETE.

Vibration produced by electric motor shown in Fig. 1. Compression tests of 6 x 12-in. cylinders. Age, 28 days. Data from Table 3.

TABLE 4.—EFFECT OF JIGGING ON THE STRENGTH OF CONCRETE.

(Concrete of Different Mixes and Consistencies.)

Values platted in Fig. 5.

All cylinders were molded by the standard method before vibrating on machine shown in Fig. 2. (100 r.p.m.; 0.1-in. drop; jigged 20 sec.)

Compression tests of 6 x 12-in. cylinders.

Age at test 28 days; stored in damp sand; tested damp.

Aggregate—Sand and pebbles from Elgin, Ill.

Each value is the average of 5 tests made on different days.

Ref. No.	Mix by Vol.	Aggregate.		Relative Consistency.	Water-Ratio to Volume of Cement.	Compressive Strength.		
		Size.	F. M.			Standard Method, lb. per sq. in.	Standard Method Plus Jigging, lb. per sq. in.	Jigged Concrete, Per cent of Standard.
75	1:7	0–1½	5.75	0.70	0.757	1270	1560	123
76	...	.....	....	0.80	0.866	1650	1910	116
77	...	.....	....	0.90	0.974	1730	2000	116
78	...	.....	....	1.00	1.081	1710	1800	105
79	...	.....	....	1.10	1.190	1480	1460	99
80	...	.....	....	1.25	1.353	1140	1110	97
						Av. 1500	1640	109
81	1:5	0–1½	5.75	0.70	0.612	1650	2020	122
82	...	.....	....	0.80	0.700	2560	2870	112
83	...	.....	....	0.90	0.788	2920	3120	107
84	...	.....	....	1.00	0.875	2840	2630	93
85	...	.....	....	1.10	0.962	2120	2040	96
86	...	.....	....	1.25	1.085	1580	1560	99
						Av. 2280	2370	104
87	1:4	0–1½	5.75	0.70	0.540	1690	2160	128
88	...	.....	....	0.80	0.618	3760	3600	96
89	...	.....	....	0.90	0.695	3750	3710	99
90	...	.....	....	1.00	0.777	3510	3190	91
91	...	.....	....	1.10	0.849	2890	2520	87
92	...	.....	....	1.25	0.965	1880	1750	93
						Av. 2910	2820	97
93	1:3	0–1½	5.75	0.70	0.469	2160	2640	122
94	...	.....	....	0.80	0.536	4050	4410	109
95	...	.....	....	0.90	0.603	4530	4800	106
96	...	.....	....	1.00	0.670	4050	3920	97
97	...	.....	....	1.10	0.737	3420	3470	101
98	...	.....	....	1.25	0.838	2680	2420	90
						Av. 3480	3610	104

TABLE 5.—EFFECT OF JIGGING ON THE STRENGTH OF CONCRETE.

(Using aggregates of different size and grading.)

Values from tests platted in Figs. 7 and 8.

All cylinders were molded by the standard method before jigging on machine shown in Fig. 2. (100 r.p.m.; 0.1-in. drop; jigged 20 sec.)

Compression tests of 6 x 12-in. cylinders.

Age at test 28 days; stored in damp sand; tested damp.

Aggregates—Sand and pebbles from Elgin, Ill., except where otherwise noted.

Each value is the average of 5 tests made on different days.

Variation in grading in last two groups was produced by mixing different percentages of sand and coarse aggregate.

Ref. No.	Mix by Vol.	Aggregate.		Relative Consistency.	Water-Ratio to Volume of Cement.	Compressive Strength.		
		Size.	F. M.			Standard Method, lb. per sq. in.	Standard Method Plus Jigging, lb. per sq. in.	Jigged Concrete, Per cent of Standard.
123	1:5	0–28	1.20	1.00	1.410	360	400	111
124	...	0–14	2.25	....	1.290	510	580	114
125	...	0–8	2.64	....	1.245	500	650	130
126	...	0–4	3.06	....	1.195	950	1060	112
127	...	0–⅜	4.04	....	1.080	1540	1270	82
128	...	0–¾	5.02	....	0.960	2290	2060	90
84	...	0–1½	5.75	....	0.875	2840	2630	93
129	1:3	0–28	1.20	1.00	0.990	680	860	126
130	...	0–14	2.25	....	0.920	1060	1360	128
131	...	0–8	2.64	....	0.890	1520	1720	113
132	...	0–4	3.06	....	0.860	2080	2020	97
133	...	0–⅜	4.04	....	0.790	3050	2930	96
134	...	0–¾	5.02	....	0.720	3840	3420	89
96	...	0–1½	5.75	....	0.670	4050	3920	97
47	1:5	0–1½	4.00	1.00	1.085	1450	1460	101
48	...	.....	5.00	....	0.960	2160	1960	91
49	...	.....	5.50	....	0.910	2500	2250	90
50	...	.....	5.75	....	0.875	2980	2360	79
51	...	.....	6.00†	....	0.845	2710	2630	97
52	...	.....	6.25†	....	0.810	2610	2520	96
53	...	.....	6.50†	....	0.785	2110	2010	95
54*	1:5	0–1½	4.00	1.00	1.085	1180	1390	118
55*	...	.....	5.00	....	0.960	2030	1850	91
56*	...	.....	5.50	....	0.910	2450	2290	93
57*	...	.....	5.75	....	0.875	2630	2430	92
58*	...	.....	6.00†	....	0.845	2420	2500	103
59*	...	.....	6.25†	....	0.810	1960	1760	90
60*	...	.....	6.50†	....	0.785	1550	1460	94

* Crushed limestone for coarse aggregate.

† These aggregates are too coarse for the quantity of cement used. See discussion on page 77.

TABLE 6.—EFFECT OF JIGGING ON THE STRENGTH OF CONCRETE.

All cylinders were molded by the standard method before jigging on machine shown in Fig. 2.
Compression tests of 6 x 12-in. cylinders.
Age at test 28 days; stored in damp sand; tested damp.
Aggregates—Sand and pebbles from Elgin, Ill. (graded 0–1½ in.).
Each value is the average of 5 tests made on different days.

Ref. No.	Mix by Volume.	F. M. of Aggregate.	Relative Consistency.	Water-Ratio to Volume of Cement.	Compressive Strength. lb. per sq. in.	Compressive Strength. Per cent of Standard.	Treatment of Concrete.
			Effect of Height of Drop. 100 r.p.m., for 20 sec. (See Fig. 9).				
31, 51, 84	1:5	5.75	1.00	0.875	2780*	100	Standard method of molding.
17	...	....	....	....	2340	84	Drop .02 in.
18	...	....	....	....	2480	89	Drop .05 in.
19	...	....	....	....	2250	81	Drop .10 in.
20	...	....	....	....	2520	91	Drop .20 in.
21	...	....	....	....	2640	95	Drop .30 in.
22	...	....	....	....	2460	88	Drop .50 in.
			Effect of Rate of Jigging, 0.1-in. drop, for 20 sec. (See Fig. 10).				
31, 51, 84	1:5	5.75	1.00	0.875	2780*	100	Standard method of molding.
12	...	....	....	....	2640	95	Jigged at 30 r.p.m.
13	...	....	....	....	2490	90	Jigged at 50 r.p.m.
14	...	....	....	....	2470	89	Jigged at 75 r.p.m.
15	...	....	....	....	2520	91	Jigged at 100 r.p.m.
16	...	....	....	....	2420	87	Jigged at 150 r.p.m.
			Effect of Time Jigged at 100 r.p.m., 0.1-in. drop (See Fig. 11).				
31, 51, 84	1:5	5.75	1.00	.875	2780*	100	Standard method of molding.
32	...	....	....	....	2440	88	Jigged for 5 sec.
33	...	....	....	....	2260	81	Jigged for 10 sec.
34	...	....	....	....	2390	86	Jigged for 20 sec.
35	...	....	....	....	2320	83	Jigged for 30 sec.
36	...	....	....	....	2251	81	Jigged for 45 sec.
37	...	....	....	....	2190	79	Jigged for 1 min.
38	...	....	....	....	2090	75	Jigged for 2 min.
39	...	....	....	....	2170	78	Jigged for 3 min.
			Miscellaneous Jigging Tests, 100 r.p.m., 0.1-in. drop, jigged 20 sec. (See Fig. 12).				
31, 51, 84	1:5	5.75	1.00	.875	2780*	100	Standard method of molding.
40	...	....	....	....	2780	100	Standard method of molding, jigged with 30-lb. weight on top. (See Figs. 9, 10 and 11.)
15, 19, 34	...	....	....	....	2390	86	Jigged immediately.
41	...	....	....	....	2550	92	Stood 1 hr. before jigging.
42	...	....	....	....	2940	106	Stood 2 hr. before jigging.
43	...	....	....	....	2950	106	Stood 3 hr. before jigging.
44	...	....	....	....	3000	108	Stood 4 hr. before jigging.
45	...	....	....	....	2870	103	Stood 6 hr. before jigging.
46	...	....	....	....	2770	100	Molded by standard method on jigging machine while in operation.

* Average of 15 tests made on different days. See Table 2.

Table 7.—Effect of Pressure on the Strength of Concrete.

All cylinders were molded by standard method before pressure was applied. Pressures higher than 50 lb. per sq. in. were applied by a testing machine, as shown in Fig. 3.

Ref. No.	Mix by Volume.	F. M. of Agg.	Relative Consistency.	Water-Ratio. As Mixed.	Water-Ratio. After Pressure.	Compressive Strength. lb. per sq. in.	Compressive Strength. Per cent of Standard.	Treatment of Concrete.
					Pressure Applied 15 Minutes.			
31, 51, 84	1:5	5.75	1.00	0.875	0.875	2780*	100	Standard method of molding.
23	...	....	....	.....	0.842	3140	113	Pressure, 2 lb. per sq. in.
24	...	....	....	.....	0.845	3000	108	" 5 lb. per sq. in.
25	...	....	....	.....	0.831	2930	105	" 10 lb. per sq. in.
26	...	....	....	.....	0.816	2900	104	" 25 lb. per sq. in.
27	...	....	....	.....	0.820	3130	113	" 50 lb. per sq. in.
28	...	....	....	.....	0.810	3470	125	" 100 lb. per sq. in.
29	...	....	....	.....	0.785	3360	121	" 200 lb. per sq. in.
30	...	....	....	.....	0.751	3590	129	" 500 lb. per sq. in.
				Av.	0.819	3150	113	
					Pressure Applied 1 Hour.			
31, 51, 84	1:5	5.75	1.00	0.875	0.875	2780*	100	Standard method of molding.
23	...	....	....	.....	0.845	2920	105	Pressure, 2 lb. per sq. in.
24	...	....	....	.....	0.824	2830	102	" 5 lb. per sq. in.
25	...	....	....	.....	0.839	2950	106	" 10 lb. per sq. in.
26	...	....	....	.....	0.834	2880	104	" 25 lb. per sq. in.
27	...	....	....	.....	0.821	3120	112	" 50 lb. per sq. in.
28	...	....	....	.....	0.810	3190	115	" 100 lb. per sq. in.
29	...	....	....	.....	0.781	3790	136	" 200 lb. per sq. in.
30	...	....	....	.....	0.756	3470	125	" 500 lb. per sq. in.
				Av.	0.820	3100	112	
					Pressure Applied 4 Hours.			
31, 51, 84	1:5	5.75	1.00	0.875	0.875	2780*	100	Standard method of molding.
23	...	....	....	.....	0.855	2880	104	Pressure, 2 lb. per sq. in.
24	...	....	....	.....	0.845	2980	107	" 5 lb. per sq. in.
25	...	....	....	.....	0.820	3200	115	" 10 lb. per sq. in.
26	...	....	....	.....	0.840	2900	104	" 25 lb. per sq. in.
27	...	....	....	.....	0.821	2990	108	" 50 lb. per sq. in.
28	...	....	....	.....	0.795	3270	118	" 100 lb. per sq. in.
29	...	....	....	.....	0.767	3350	121	" 200 lb. per sq. in.
30	...	....	....	.....	0.750	3680	132	" 500 lb. per sq. in.
				Av.	0.819	3120	112	
					Pressure Applied 16 Hours.			
31, 51, 84	1:5	5.75	1.00	0.875	0.875	2780*	100	Standard method of molding.
23	...	....	....	.....	0.834	3060	110	Pressure, 2 lb. per sq. in.
24	...	....	....	.....	0.824	2710	97	" 5 lb. per sq. in.
25	...	....	....	.....	0.815	2630	95	" 10 lb. per sq. in.
26	...	....	....	.....	0.846	2670	96	" 25 lb. per sq. in.
27	...	....	....	.....	0.798	3320	119	" 50 lb. per sq. in.
28	...	....	....	.....	0.795	3330	120	" 100 lb. per sq. in.
29	...	....	....	.....	0.794	3410	123	" 200 lb. per sq. in.
30	...	....	....	.....	0.766	3420	123	" 500 lb. per sq. in.
				Av.	0.816	3040	109	

TABLE 7.—*Con'inued.*

Ref. No.	Mix by Volume.	F. M. of Agg.	Relative Consistency	Water-Ratio		Compressive Strength		Treatment of Concrete.
				As Mixed.	After Pressure.	lb. per sq. in.	Per cent of Standard.	
Grand Average of all Times of Application of Pressure (See Figs. 13 to 16).								
31,51,84	1:5	5.75	1.00	0.875	0.875	2780*	100	Standard method of molding.
....	...	....	....	.....	0.844	3000	108	Pressure, 2 lb. per sq. in.
....	...	....	....	.....	0.834	2880	104	" 5 lb. per sq. in.
....	...	....	....	.....	0.826	2930	105	" 10 lb. per sq. in.
....	...	....	....	.....	0.834	2840	102	" 25 lb. per sq. in.
....	...	....	....	.....	0.815	3140	113	" 50 lb. per sq. in.
....	...	....	....	.....	0.802	3320	119	" 100 lb. per sq. in.
....	...	....	....	.....	0.782	3480	125	" 200 lb. per sq. in.
....	...	....	....	.....	0.756	3540	127	" 500 lb. per sq. in.

* Average of 15 tests made on different days, see Table 2.

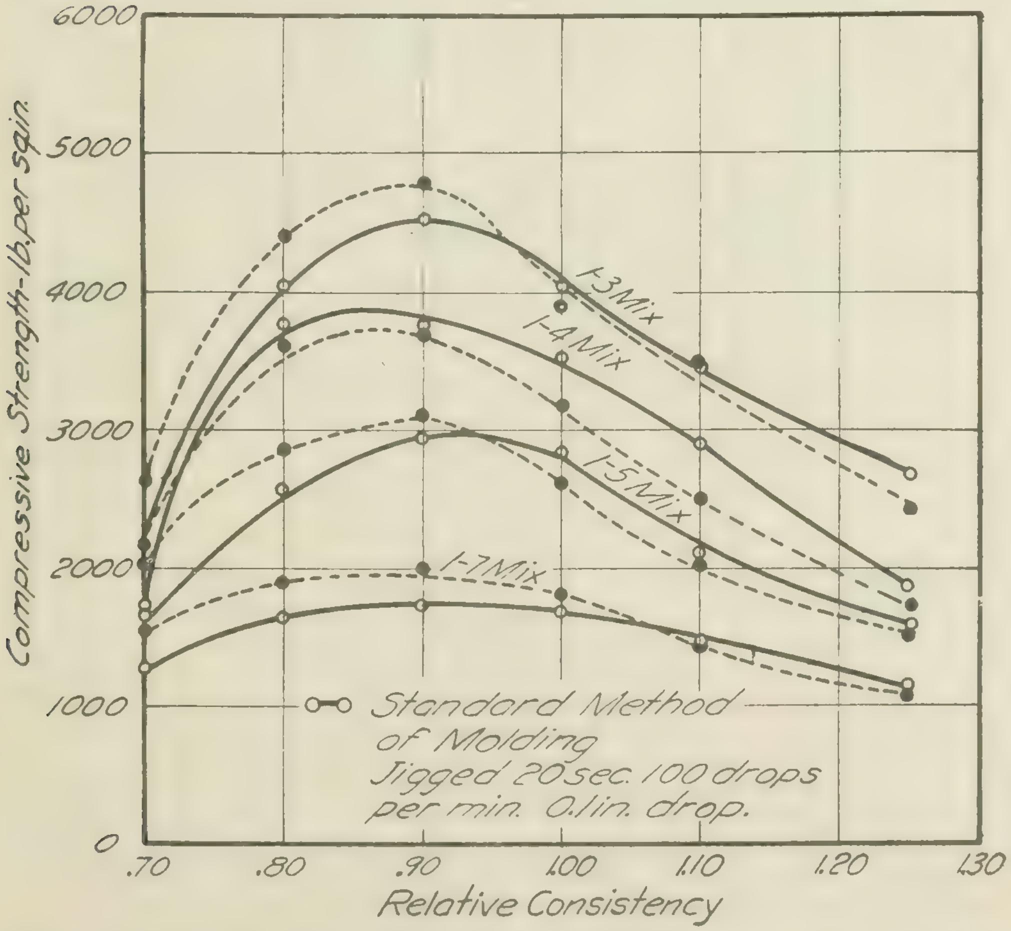


FIG. 5.—EFFECT OF CONSISTENCY ON THE STRENGTH OF JIGGED CONCRETE.

Specimens molded by standard method and parallel sets jigged on machine shown in Fig. 2. Compression tests of 6 x 12-in. cylinders. Age 28 days. Data from Table 4.

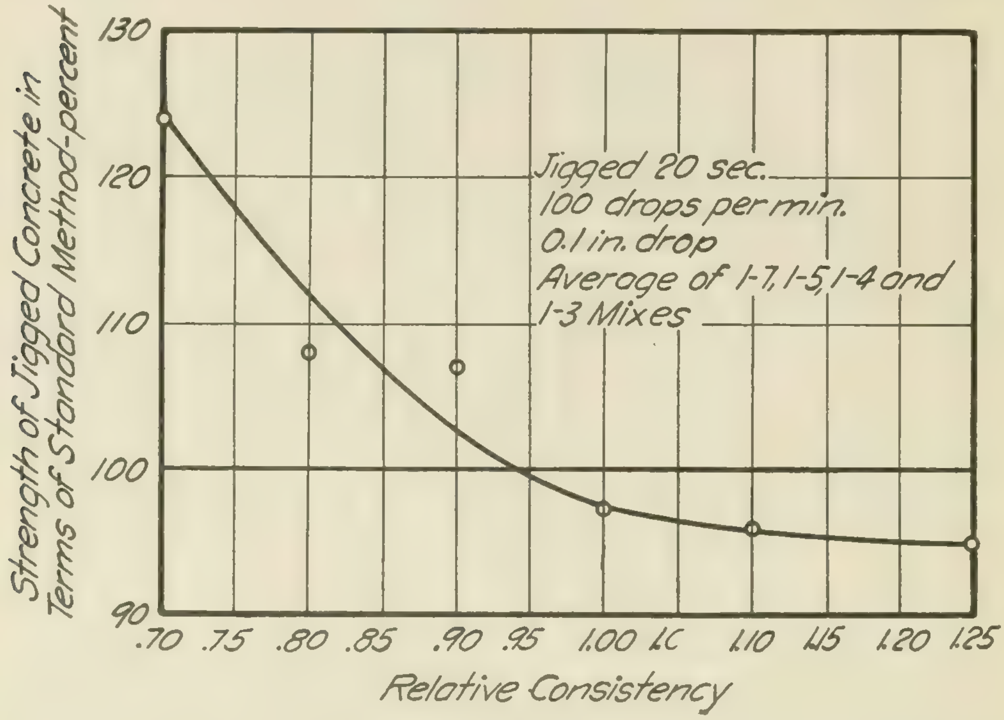


FIG. 6.—EFFECT OF CONSISTENCY ON THE STRENGTH OF JIGGED CONCRETE.

Each value is the average of 20 tests, 5 each from 4 different mixes in Table 4.

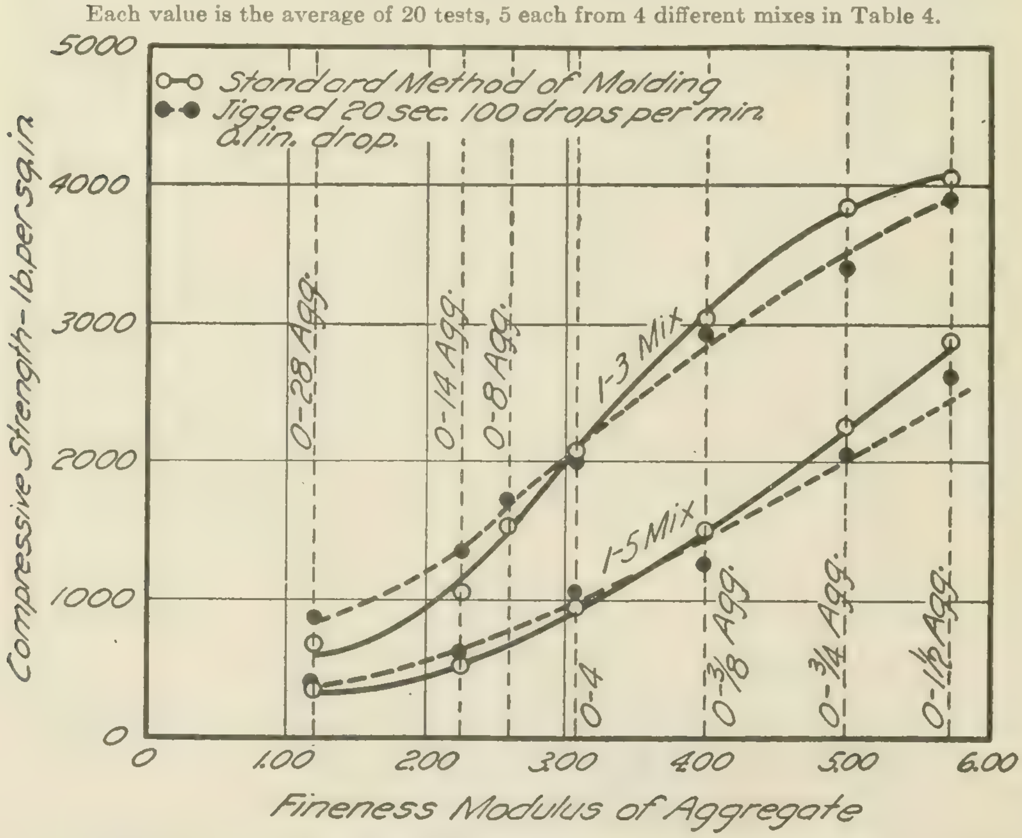


FIG. 7.—EFFECT OF SIZE OF AGGREGATE ON THE STRENGTH OF JIGGED CONCRETE

Compression tests of 6 x 12-in. cylinders. Age 28 days. Data from Table 5.

indicate that hand-puddling (if thoroughly done) is just as effective as vibration in placing concrete. In other words, concrete which completely fills the form is not improved by vibration. This should not be construed as meaning that vibration is not effective in causing concrete to find its way into intricate form work and around the reinforcing bars.

The effect of jigging is shown in Tables 4 to 6, and Figs. 5 to 12. The jigging method used was one which would be applicable to concrete products plants. The tests show that in general jigging of concrete which has been placed by puddling is injurious to the strength. The improvement for the small-sized aggregates is probably due to the fact that the puddling method is not so satisfactory for this condition.

In comparing the tests of crushed limestone and pebbles in Table 5 it should be noted that the three coarsest gradings in each group (fineness modulus 6.00 and over) are too coarse for this mix, consequently the concrete strengths fall off. The extremely coarse mixtures cause a much greater reduction in the strength of the concrete made from crushed stone than from gravel. For this reason it is not proper to draw conclusions from a comparison of average values from the two types of coarse aggregate. If we confine our attention to the ordinary range in aggregate grading (fineness modulus 5.00 to 5.75) we find little difference in strength of concrete from the two materials. For these conditions the average strength with pebbles is 2550 lb. per sq. in. for the standard method of hand-puddling and 2190 after jigging; corresponding values for crushed stone are 2370 and 2190 lb. per sq. in.

The tests of concrete setting under pressure are of interest in that they reveal the reason for increased strength due to this treatment. The concrete strength is increased due to the fact that some of the original mixing water is forced out. The higher the pressure, the more water is removed, hence the higher the strength.

Attention is called to the unusual uniformity of the results of the tests in this series, as illustrated by the fact that in the diagrams the points fall on smooth curves. There are only a few instances in which there is any appreciable deviation from this rule.

SUMMARY AND CONCLUSIONS.

The tests gave conclusive results on many phases of the effect of vibration, jigging and pressure. In some instances the effect is entirely different from what accepted opinion would suggest. Following is a brief summary of the tests:

Effect of Puddling and Tamping (Table 2).

1. Varying the number of strokes from 12 to 50 on each 4-in. layer in the standard method of hand-puddling with a $\frac{5}{8}$-in. bar had little influence on the compressive strength of ordinary plastic concrete.

2. In general, the tamping methods used gave lower strengths than hand-puddlng.

3. A tamper of large diameter for a given weight was less effective than one of small diameter.

4. Increasing the thickness of the layer from 4 to 6 in. caused a falling-off in strength of about 12 per cent for tamped concrete.

5. Tamping or puddling the first 4-in. layer only, caused a falling off in strength of 10 to 13 per cent.

6. Striking the metal form with a steel bar after the completion of molding by standard method had no effect on the strength of concrete.

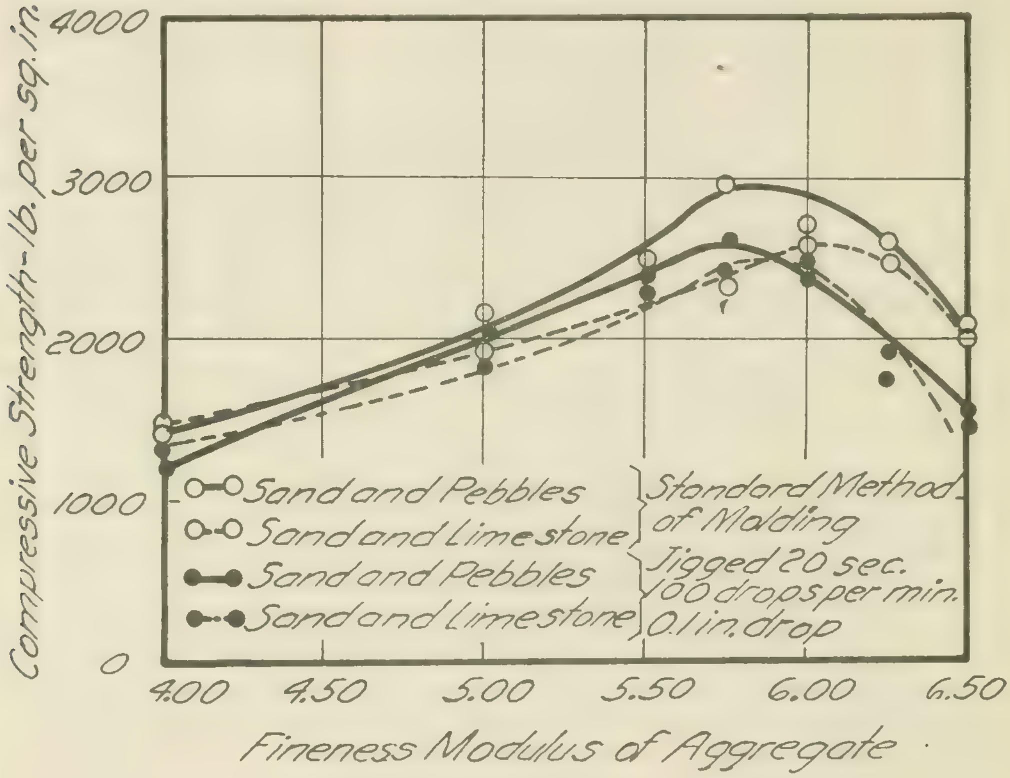


FIG. 8.—EFFECT OF GRADING OF AGGREGATE ON THE STRENGTH OF JIGGED CONCRETE.

Specimens molded by standard method and parallel sets jigged on machine shown in Fig. 2. Compression tests of 6 x 12-in. cylinders. 1:5 mix. Age, 28 days. All aggregate graded 0-1½ in. Data from Table 5.

7. The "standard" method of hand-puddling using 25 strokes with a ⅝-in. steel bar for each 4-in. layer of concrete in a 6 x 12-in. cylinder is recommended for laboratory tests of concrete.

Effect of Vibration with Electric Hammer.

8. Vibration of the specimen after molding by means of an electric hammer running at 1000 r.p.m. had little influence on the strength of the puddled concrete up to a period of about 30 seconds. If continued, there was a steady falling off in strength; after 45 to 60 seconds the strength was only 90 per cent. of that produced by the standard method of puddling. (Table 3, and Fig. 4).

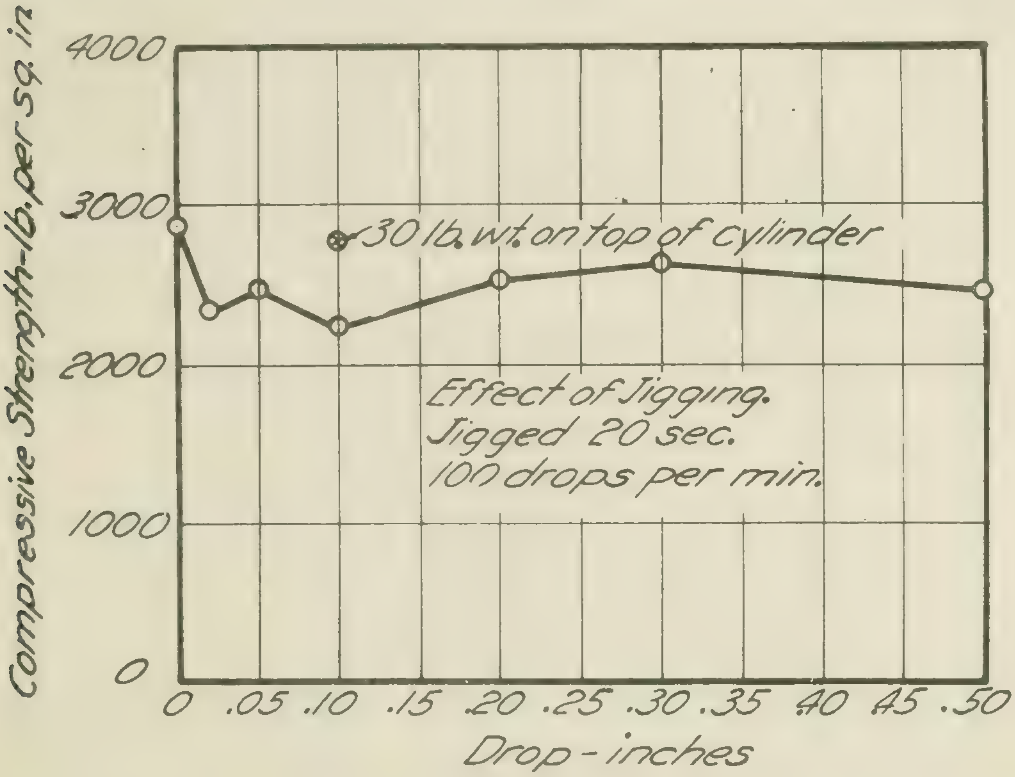


FIG. 9.—EFFECT OF HEIGHT OF DROP IN JIGGING TESTS.
Compression tests of 6 x 12-in. cylinders. Age 28 days. Data from Table 6.

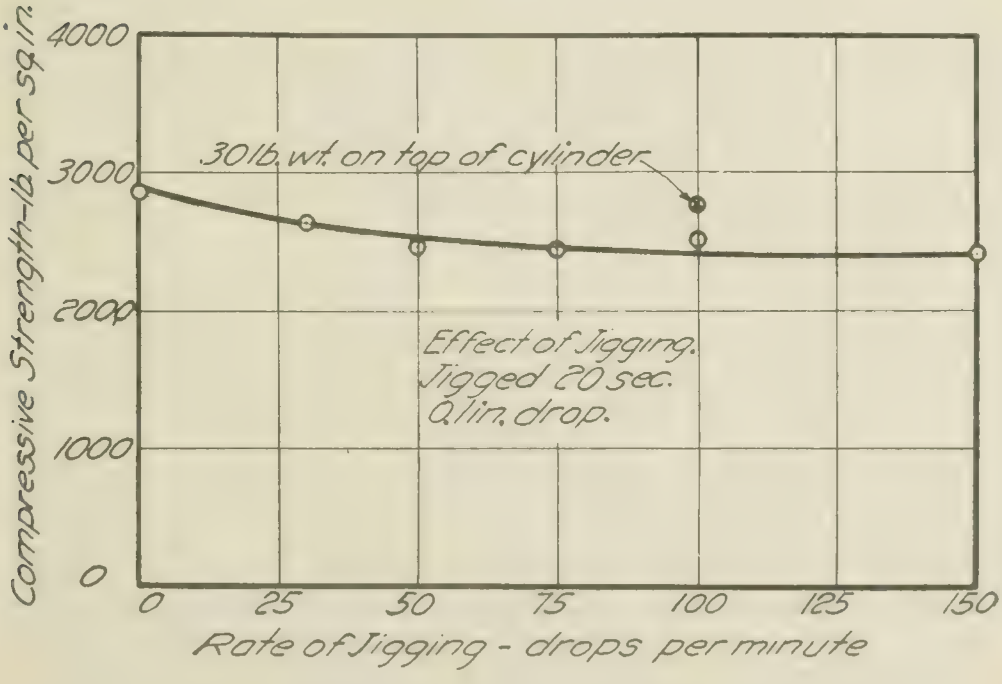


FIG. 10.—EFFECT OF RATE OF JIGGING ON THE STRENGTH OF CONCRETE.
Compression tests of 6 x 12-in. cylinders. 1 : 5 mix. Age 28, days. Data from Table 6.

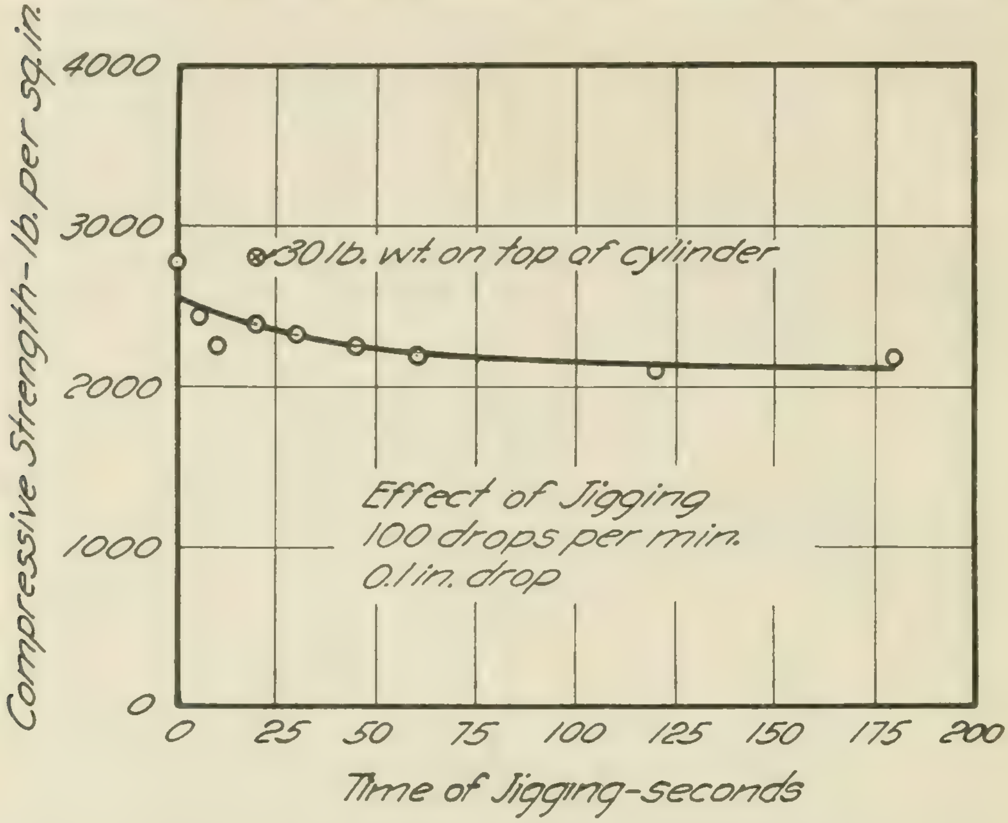


FIG. 11.—EFFECT OF DURATION OF JIGGING ON THE STRENGTH OF CONCRETE.

Compression tests of 6 x 12-in. cylinders. 1 : 5 mix. Age. 28 days. Data from Table 6.

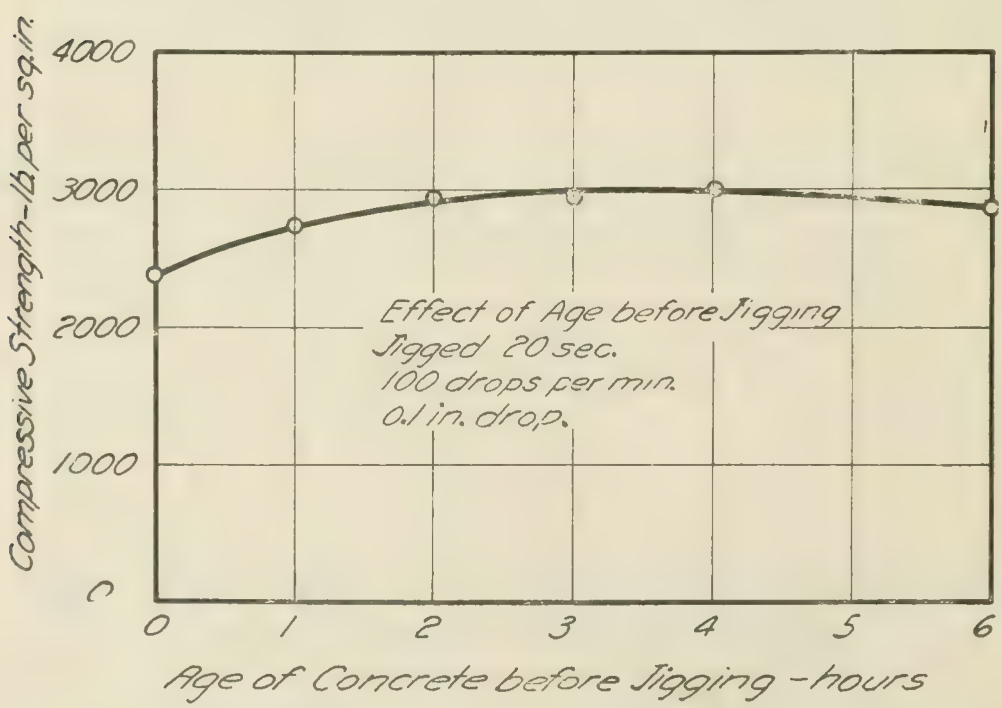


FIG. 12.—EFFECT OF AGE OF CONCRETE BEFORE JIGGING.

Compression tests of 6 x 12-in. cylinders. 1 : 5 mix. Age. 28 days. Data from Table 6.

EFFECT OF JIGGING.

9. In general, jigging in any manner with the apparatus used reduced the compressive strength of the concrete regardless of the height of drop, rate or duration of treatment. Exceptions were found in the dry mixes and those made of aggregates of the smaller sizes. (Tables 4 to 6; Figs. 5 to 12.)

10. There was little difference in the effect of jigging due to the quantity of cement used. (Fig. 5.)

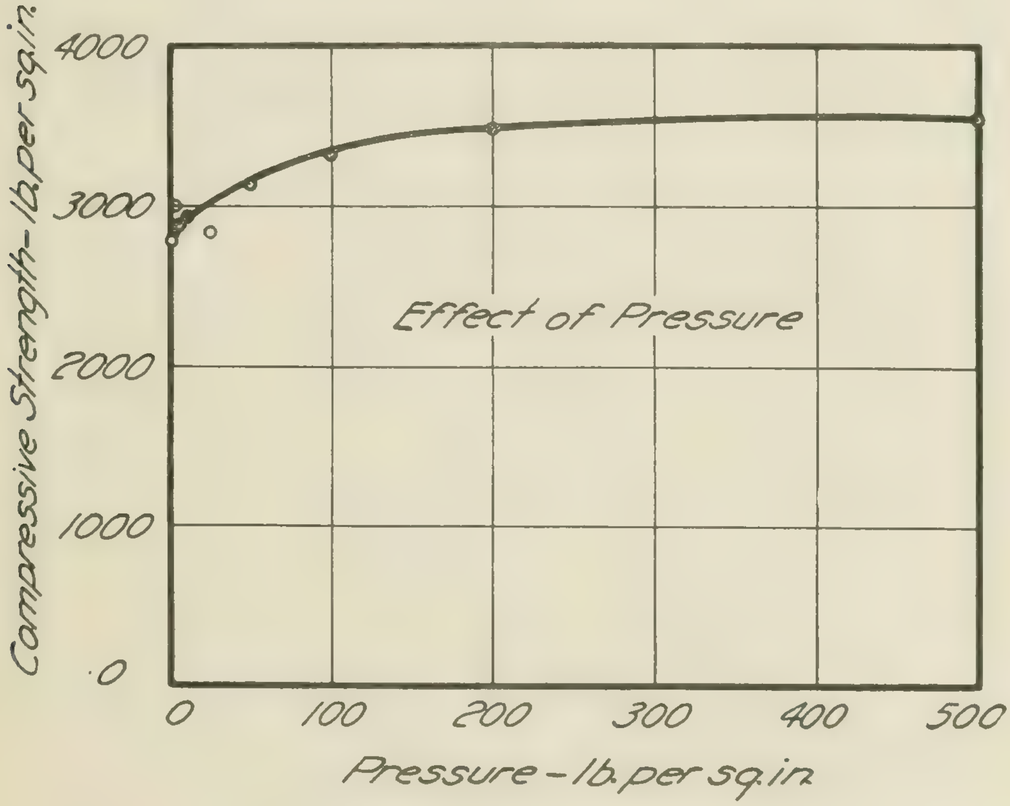


FIG. 13.—EFFECT OF PRESSURE ON THE STRENGTH OF CONCRETE.

Compression tests of 6 x 12-in. cylinders. 1 : 5 mix. Age, 28 days. Pressure applied immediately after molding. Each point represents the average of 20 tests, 5 each from 4 different times of application of pressure ranging from 15 min. to 16 hr. Data from Table 7

11. In the very dry mixes the strength, due to jigging for 20 seconds, was increased about 25 per cent. (Figs. 5 and 6.)

12. The wetter mixes (relative consistency 1.10 to 1.25) were reduced in strength 3 to 6 per cent by jigging. (Figs. 5 and 6.)

13. Pebbles and crushed limestone as coarse aggregate gave essentially he same results in the jigging tests. (Fig. 8.)

14. The concretes for finer aggregates showed a material increase in strength with jigging in both 1:5 and 1:3 mixes. (Fig. 7.)

15. For aggregate coarser than about $\frac{3}{8}$ in., jigging reduced the strength from 3 to 10 per cent. (Fig. 7.)

16. The grading of the aggregates (for a given maximum size) had little influence on the effect of jigging. (Fig. 8.)

17. The greater the drop the greater the reduction in strength for 1:5 concrete. For a drop of $\frac{1}{2}$ in. the strength was reduced 12 per cent. (Fig. 9.)

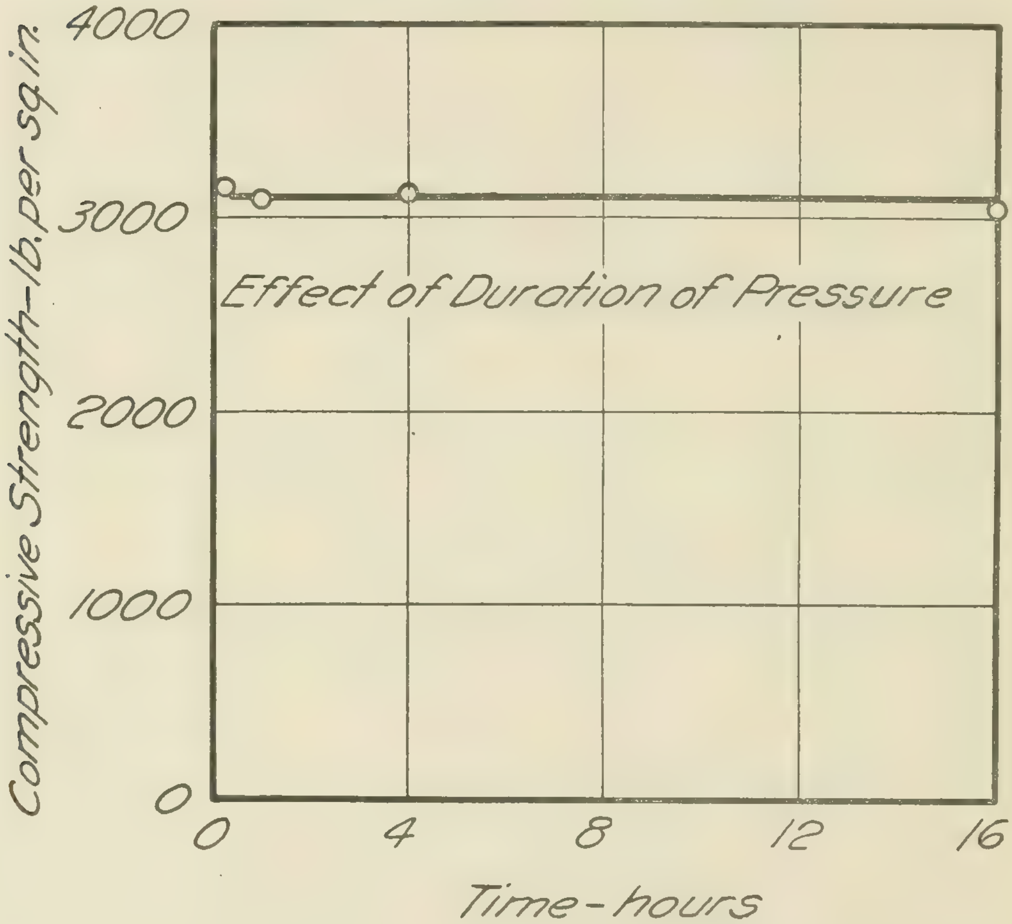


FIG. 14.—EFFECT OF DURATION OF PRESSURE ON THE STRENGTH OF CONCRETE.

Compression tests of 6 x 12-in. cylinders. 1 : 5 mix. Age, 28 days. Pressure applied immediately after molding specimen. Each point represents the average of 45 tests, 5 each from 9 different pressures ranging from 2 to 500 lb. per sq. in. Data from Table 7.

18. The faster the rate of jigging the lower the strength of 1:5 concrete. Using $1\frac{1}{2}$-in. aggregate at 150 r. p. m. the strength was reduced about 13 per cent. (Fig. 10.)

19. The strength of 1:5 concrete fell off rapidly with the duration of jigging. After 2 to 3 minutes jigging the strength was reduced about 20 per cent. as compared with standard method of hand-puddling. (Fig. 11.)

20. Allowing the concrete to stand for a period of time before jigging, increased the strength to a slight extent. The maximum increase was found at 2 to 4 hr. (Fig. 12.)

21. The application of a pressure of 1 lb. per sq. in. during the jigging process (equivalent to a head of 1 ft. of fresh concrete) gave the same strength as standard hand-puddling. (Figs. 9, 10 and 11).

22. Molding the cylinders by the standard method on the jigging table while it was in motion, gave the same strength as standard hand-puddling without jigging. (Table 6.)

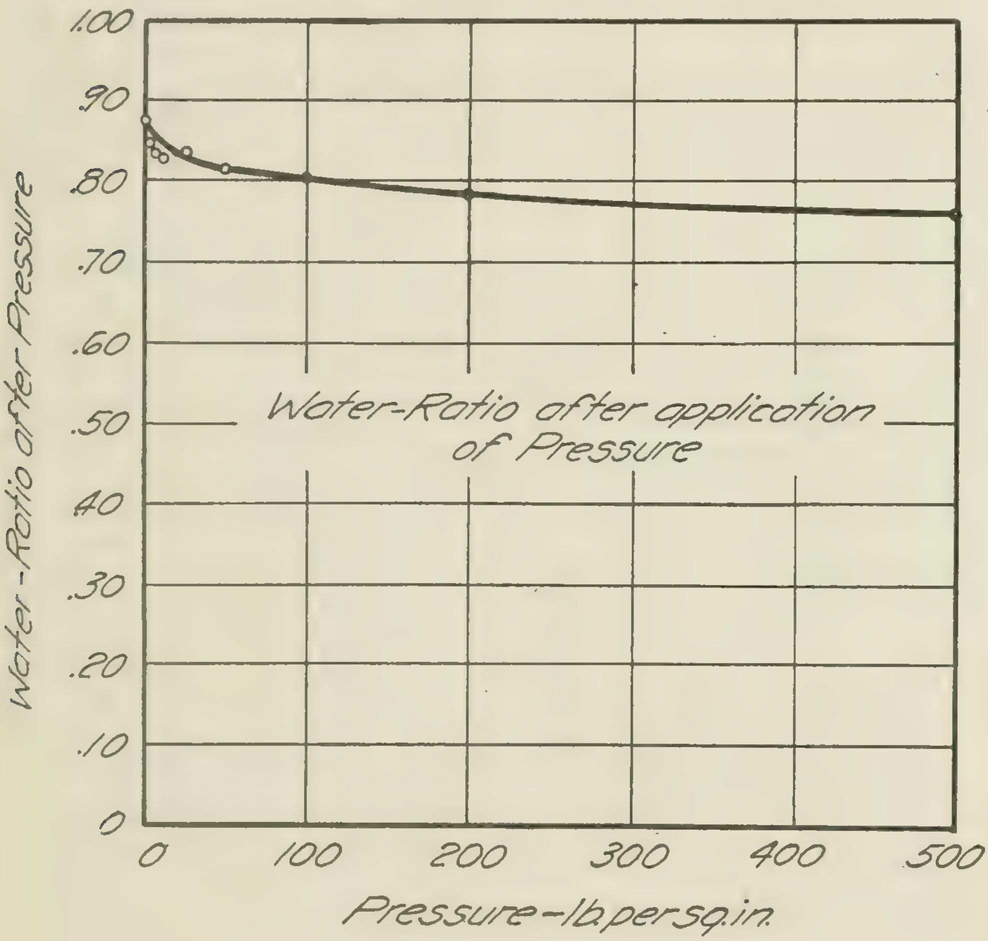


FIG. 15.—WATER-RATIO OF CONCRETE AFTER PRESSURE.

Each point represents the average of 20 tests, 5 each from 4 different times of application of pressure. Data from grand average values in Table 7.

Effect of Pressure.

23. The compressive strength of concrete was increased by pressure applied immediately after molding. For pressure of 200 to 500 lb. per sq. in. the increase was 20 to 35 per cent. (Fig. 13).

24. The duration of pressure as between 15 min. and 16 hr. produced no difference in strength. (Fig. 14.)

25. There was a steady reduction in the water-ratio of the concrete with the application of pressure. (Fig. 15.)

26. The application of pressure increased the strength of concrete in accordance with the quantity of mixing water expelled. (Fig. 16.)

27. The tests of concrete subjected to pressure showed the usual relation between compressive strength and water-ratio. The strength is increased *because the water is expelled.* In other words, pressure produces a drier concrete, and consequently gives higher strength.* This makes it clear why the duration of pressure has no influence on the result.

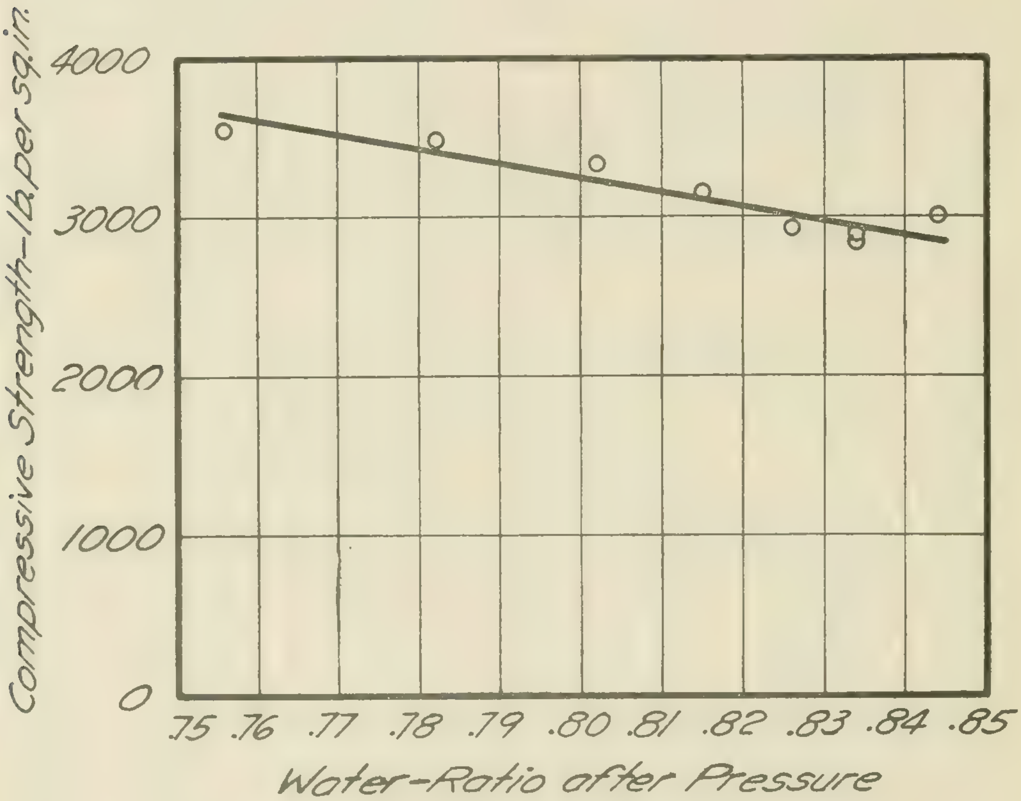


FIG. 16.—EFFECT OF QUANTITY OF MIXING WATER ON THE STRENGTH OF CONCRETE.

Compression tests of 6 x 12-in. cylinders. Age, 28 days. Water-ratio determined after application of pressure. Each point represents the average of 45 tests, 5 each from 9 different pressures. Data from grand average values in Table 7. The narrow range in water-ratio gives a straight line relation for these tests. The water-strength relation is represented by a curved line as shown in Bulletin 1, Structural Materials Research Laboratory.

FURTHER DISCUSSION OF VIBRATION, JIGGING AND PRESSURE TESTS.

The indications of the vibrations and jigging tests should not be misinterpreted. The tests show that *after the concrete is properly placed* these methods of treatment do no good and may be harmful if too severe or too long continued. However, there can be no doubt of the value of such methods

* For effect of consistency on the strength of concrete, see Bulletin 1 referred to above; and "Effect of Time of Mixing Concrete," Proceedings, American Concrete Institute, 1918.

for getting concrete into place in intricate forms and around reinforcing bars. The tests are of value in showing that this is the only desirable function of such treatments. The tests under Ref. No. 149 (Table 2) show the ill effects of lack of compactness in the concrete. Here the strength was reduced 13 per cent due to failure to tamp or puddle the top 9 in. of the cylinder. It is impracticable to duplicate in a compression test piece the performance of air hammers and other similar methods of vibrating when used on reinforced concrete work.

The tests show that with jigging high strength may be secured with drier mixes than would be feasible otherwise. It is a matter of common experience that concrete of drier consistency (and consequently higher strength) can be placed by means of jigging or vibration than would be possible by the usual methods.

The roller method of finishing concrete roads, walks and floors is an interesting example of a combination of slight vibration and pressure accompanied by the removal of excess water. Transverse tests on concrete made in this Laboratory showed a marked increase in strength of the rolled slabs as compared with similar slabs without rolling.*

It is clear from these tests that if tamping, vibration or pressure on fresh concrete are to be effective in increasing its strength three factors must be kept in mind:

(1) We must take advantage of the fact that with these methods the concrete can be placed and finished dryer than with ordinary methods.

(2) Excess water which is brought to the surface must be removed.

(3) We must take advantage of the fact that aggregate of a coarser grading may be used when such methods are employed than would be practicable otherwise.

The advantages to be gained under (3) are due to the fact that up to a certain point a plastic mix can be secured with a smaller quantity of water if the aggregate is as coarse as practicable. Unless these precautions are taken, tamping and vibration are of doubtful value.

* See paper of A. N. Johnson, Proc. Am. So. Testing Materials, 1917, Part II, p. 378.

DISCUSSION.

President W. K. Hatt in the Chair.

Mr. Davis.

Mr. Watson Davis.—I would like to call attention to several relations between water-ratio and compressive strength that are found in Table 5. In the second group of tests there is a water-ratio of 0.920 and a strength of 1,060 lb. per sq. in., while in the third group there is a water-ratio of 0.910 and a corresponding strength of 2,500 lb. per sq. in. That is a difference of 1,500 lb. per sq. in. between concretes having essentially the same ratio. Just below the first example in the second group there is a water-ratio of 0.790 and a compressive strength of 3,050 lb. In the third group there is a water-ratio of 0.785 with a compressive strength of 2,110 lb. per sq. in., a difference of 1,000 lb. per sq. in. between two of the same water-ratio. Yet I understand from Professor Abrams, water-ratio is the governing essential in strength, and that concretes of the same water-ratio should have the same strength.

Mr. Abrams.

Mr. D. A. Abrams.—It should be pointed out that in this particular investigation we were studying the effect of size in the grading of aggregates. In other words, these aggregates were deliberately made very much coarser toward the bottom of the group than would ever be feasible or practicable for use in concrete, and consequently we have never claimed that our water-ratio relations hold under those conditions. In other words, in the third group of Table 5, the last three values would probably be what we call over the peak. If you will refer to Figs. 7 and 8, it will be quite evident that that is the case. Fig. 8 shows this peak effect. Then the strength drops off very rapidly. That does not invalidate the water-ratio period, but simply corroborates what is said in other instances—that the grading of aggregates become too coarse, and does here become too coarse for a proper concrete mixture; consequently, we get a very low strength.

Mr. Talbot.

Mr. K. H. Talbot.—I would like to ask if the concrete in the jigging specimens was filled into a form, the top inch or two inches of which had holes around it so that the water could easily get away, whether that would not have the same effect as compressing the specimen in increasing the strength? It is usual to over-fill the forms, so that the water that is brought to the surface by creating a greater density of the concrete, is taken off. Would not the jigging have the desirable effect of taking out that excess water which is put into the form in order to make it more easy to handle the concrete?

Mr. Abrams.

Mr. Abrams.—In most of these tests the concrete was of such consistency that we did not get an excess of water on top of the specimen. We were quite surprised at that result. In fact, we expected that this jigging would bring a great deal of water to the surface, but that, however, was

not the case. Granting that we had done what Mr. Talbot suggests, I doubt very much if that would serve the purpose that he has in mind. I do know that if you make a concrete specimen with an excess of water, so that you get a quantity of free water on top, that the mere fact that you remove the water affects the make-up of the concrete below. In other words, what you apparently have is a certain head of water which is effective in distributing the particles of cement throughout the concrete; that is the thing that determines the strength that you are going to get. The mere fact that you take this water away from the top of the specimen would probably serve no useful purpose. If you could take it away from the bottom and relieve the water throughout the height, undoubtedly it would be effective. **Mr. Abrams.**

MR. RICHARD L. HUMPHREY.—On the other hand, I think it is a fact that the leaner mixtures, when vibrated, will give a dense concrete, and you can obtain by vibration methods with a lean mixture a greater strength than can be obtained with a richer mixture by hand methods. I think vibration serves the purpose not only of bringing out the excess water which creates in concrete, but also the air. **Mr. Humphrey.**

MR. TALBOT.—I would like to ask whether we could draw any conclusions from these tests in connection with the vibration of the surface of a concrete road, where the water is drawn to the surface? In tests that were made with a roller, in which the water was taken out of the concrete, it was shown that there was a considerable advantage in strength by rolling the surface and thereby removing water. Would that not apply just as much if we were able to vibrate it as the Vibrolithic Co. does, using a small gasoline motor mounted on a small truck, which gives a vibration, bringing the water to the surface and passing it off? **Mr. Talbot**

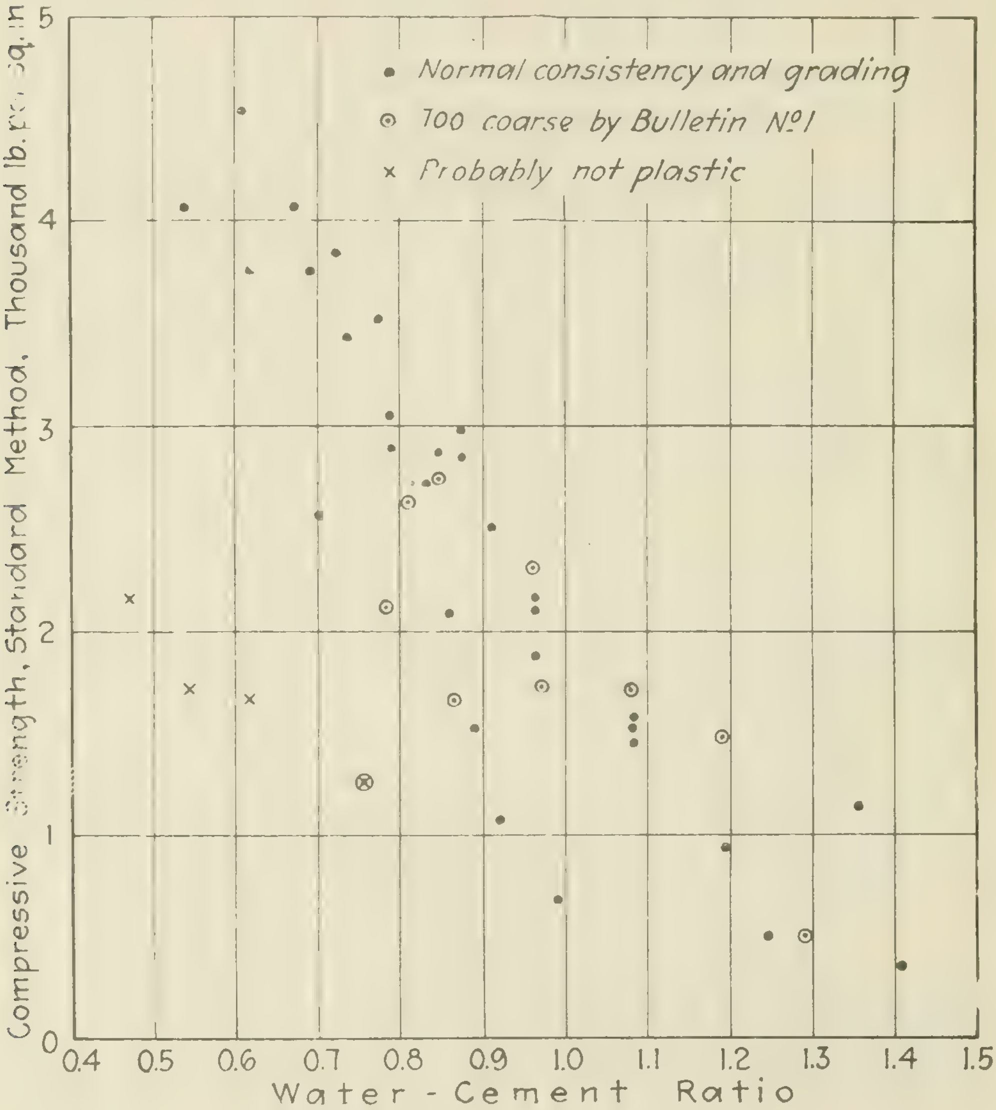
MR. ABRAMS.—There is no question of the efficacy of that method, but I think there must be a very distinct difference in the effect produced on the concrete. In other words, you are exerting the pressure from the top down in one case and in the other case you are allowing the concrete to drop of its own weight, and that dropping seems to have the effect of keeping it in continuous agitation, so that whatever benefit you may secure from the drop itself is destroyed by the rebound. I am not sure that I can explain exactly the effect, but the strength tests indicate that you do not get any beneficial result. **Mr. Abrams**

MR. HUMPHREY.—I think that is a little erroneous. The effect of rolling the vibratory machine on top of the concrete is not one of pressure, but it is the jiggling motion. It assists in shaping the aggregates into place, thereby eliminating air voids and water voids. In the government concrete ships the vibratory methods used serves purely the purpose of shaking the materials into close contact and eliminating voids. **Mr. Humphrey.**

MR. WATSON DAVIS (*by letter*).—In the oral discussion of this paper, it was pointed out that several mixes in Tables 4 and 5, which were made with practically the same water ratios, gave widely different compressive strengths when molded by his standard method. According to the theory advanced by Professor Abrams, in Bulletin 1 of Lewis Institute, **Mr. Davis.**

Mr. Davis. these concretes having the same water ratio should have the same strengths.

In the accompanying diagram the standard method compressive strengths are plotted against the water ratios given in Tables 4 and 5. Results of mixes 54 to 60, in which limestone was used as coarse aggregate, are not plotted. The mixes, which are too coarse when judged by the



SPECIMENS FROM TABLES 4 AND 5.

standards set forth by Professor Abrams in Table 3 of Lewis Institute Bulletin I are indicated, and those which probably were not of a "workable plasticity" (under 0.80 consistency) are also indicated.

It should be noted that in several cases the variation in strength of mixes with the same water ratio is as much as 1,500 lb. per sq. in., and that similar differences of less magnitude are general. Several mixes, having 0.2 to 0.5 difference in water ratio, have similar strengths and, in some cases, the mixes of higher w/c have higher strengths.

FIRE TESTS OF CONCRETE COLUMNS.*

By Walter A. Hull.†

A progress report of fire tests of concrete columns which are being conducted at the Pittsburgh laboratories of the Bureau of Standards was published in the Proceedings, of the American Concrete Institute for 1918. The tests that have been made since that report was prepared extend the scope of the investigation by including additional aggregates and new applications of fire protective material. The tests reported in 1918 showed the behavior of columns of various types with coarse aggregates of Pittsburgh gravel and West Winfield, Pa., limestone. They also gave the results of two tests of columns with the so-called Cow Bay gravel and of a number of tests of columns which had had the protective concrete supplemented by the addition of other material, applied by plastering. The more recent tests include columns from Cow Bay gravel and pure quartz gravel from Long Island and from trap rock and blast furnace slag. A number of columns, with Pittsburgh gravel, in which other protective materials have been substituted for the greater part of the protective concrete have also been tested.

The additional aggregates have been tested in round columns with vertical and spiral reinforcement and in vertically reinforced square columns. It was found, in the earlier part of the investigation, that the various aggregates show their fire-resisting characteristics more strongly in columns of these two types than in round columns without spiral reinforcement and it was therefore not considered expedient to make columns of the latter type for the further study of aggregates.

With the exception of columns cast in forms made from gypsum, all the columns were made 18 in. in diameter, if round, or 16 x 16 in. if square. In the 18-in. round columns the thickness of protective material, over the spiral, was 1½ in. The effective diameter of spirally reinforced round columns was taken as the horizontal distance between centers of the hooping. In the square columns, the vertical rods were placed at the same distance from the surface of the column as in round ones and the effective area was taken as that portion within a square with its sides located $\frac{5}{32}$ in. beyond the outside of the rods to correspond with the criterion followed in figuring the effective area of the spirally reinforced round columns. All columns were made 8 ft. 9 in. long.

Mixture and Consistency.

As in the columns previously reported, one proportion, 1:2:4, has been maintained throughout. Concrete was mixed and placed by hand. The consistency was somewhat less fluid than that ordinarily obtained with

* By permission of the Director, Bureau of Standards

† U. S. Bureau of Standards, Pittsburgh Pa.

machine mixing, being that resulting from the use of a weight of water approximately 8 per cent of that of the total weight of the dry batch. It might properly be called a quaking consistency. When first mixed, it would stand in a somewhat flattened mound without spreading out over the floor. The concrete required considerable poling in placing. There was very little tendency for water to separate and rise to the top; when the form was filled to overflowing, the overflow was mortar, for the most part, rather than separated water. Columns, whether fire tested or not, did not show a tendency to give top failures.

TEST APPARATUS.

The test apparatus consisted essentially of a gas–fired furnace which is provided with a loading equipment so that a load can be kept on the column during test and that the column can be tested for strength, while still hot, if desired. The furnace is fired with natural gas, using twelve burners of a blast type, compressed air being used to secure a quick and thorough mixture of gas and air, which burns with a short flame. Fires are kept oxidizing. Baffles are so arranged in front of the burners that the flames spread out over the walls of the furnace to some extent on entering the furnace chamber, giving fairly uniform distribution of the heat, throughout the furnace and avoiding any localized impingement on the column.

The load equipment consists of a hydraulic jack, with separate hand pump, the jack being supported above the center of the furnace by two pairs of I-beams which are held by tension rods passing through the ends of a reinforced-concrete girder which extends under the furnace, supporting the foundation for the column. The capacity of the jack is 500 tons. The capacity of the supporting steelwork is rated at 600,000 lb. With the 500-ton jack and hand test pump, the 600,000 lb. load can be applied without difficulty. The jack and test pump, with gage, have been calibrated in one of the testing machines in the engineering testing laboratory of the Bureau, and this calibration is used in the determination of the loads on the columns.

Temperatures are measured, both in the furnace and in the interior of the column, by means of iron-constantan thermocouples and a Leeds and Northrup potentiometer indicator. The couples in the column are placed before the column is cast, a special device being used to hold the couples in position during the placing of the concrete.

In all tests, the working load of the column is kept on the column during the fire test. In most of the tests the load has been increased, at the finish of the fire test, up to the ultimate strength of the column or to the capacity of the test apparatus. If the column withstands the 600,000 lb. load, it is permitted to cool, transferred to the 10,000,000 lb. machine and tested in that.

The firing of the furnace is regulated, as closely as possible, to conformance with the standard fire-test curve.

ROUND COLUMNS FROM ADDITIONAL AGGREGATES.

Date, in condensed form, giving results of tests of round columns from additional aggregates are given in Table I. In considering these results, in

connection with the results of tests published last year, it should be taken into account that most of these newer tests were made at ages slightly greater than four months, whereas the earlier ones were made on columns considerably older than these, most of them older than six months.

Columns 42 and 43 were of gravel from Long Island. This gravel was

TABLE I.

18-in. cylindrical columns.
Reinforcement: 2 per cent vertical, 8 round rods, $\frac{3}{4}$ in. diam.
1 per cent spiral, $\frac{5}{16}$ in. diam., 2 in. pitch, 2 spacers.
Effective area concrete, 168.7 sq in.
Area vertical steel, 3.53 sq. in.
Effective area column, 172 sq. in.
Working load, 141,500 lb.

	Column Number.	Maximum load, lb.		Maximum Stress, lb. per sq. in.	Maximum Temperature at End of 4 hour Fire Test, deg. C.		
		Without Fire Test.	At End of 4-hour Fire Test.		In Steel.	Midway.	At Center.
Pure quartz gravel......	42	....	*	....	990	....	....
" " "	43	....	*	....	990	350	150
Cow Bay gravel........	46	....	*	....	985	275	100
" " "	47	....	*	...	1000	250	105
Blast furnace slag......	48	....	465,000	2700	480	85	85
" " "	49	839,000	....	4870	....	....	....
" " "	50	....	393,000	2260	465	110	100
Trap rock............	54	....	417,000	2420	610	190	100
" "	55	....	516,000	3000	560	239	110
Pittsburgh gravel......	58	....	365,000	2120	620	140	100
" "	59	....	525,000	3050	410	100	90
" "	60	....	207,000	1200	810	260	90
" "	61	....	482,000	2800	185	160	110
" "	63	....	304,000	1770	590	185	90
" "	66	....	222,000	1290	765	335	150
" "	65	....	*	....	440	220	100

* Column No. 42 failed, under working load, at end of 3 hr. 32 min.
Column No. 43 failed, under working load, at end of 3 hrs.
Column No. 46 failed, under working load, at end of 3 hrs. 37 min.
Column No. 47 failed, under working load, at end of 3 hrs. 40 min.

Columns Nos. 58 and 59 were cast in gypsum forms of such dimensions as to give a protective coating of 1 in. concrete and 3 in. gypsum. Wire hoops over the gypsum on No. 59.

Columns Nos. 60 and 61 were cast in gypsum forms of such dimensions as to give $\frac{1}{2}$ in. of concrete and 2 in. of gypsum over the steel. Anchorage for the gypsum was provided in the case of No. 61.

Column No. 63 was plastered with cement plaster, over the concrete. Concrete column was 16 in. in diam., plastered so as to make 18 in. diam.

Column No. 66 was cast in a form made by covering the spiral reinforcement with metal lath and plastering on the metal lath with cement plaster.

Column No. 65 was made the same as No. 63, except that a special plaster, containing asbestos, was used.

made up almost entirely of smooth, nearly white pebbles, fairly well graded as to size. These columns behaved in a manner similar to that of the columns of the same type from Pittsburgh gravel, previously reported. The protective concrete commenced spalling early in the burn. In the case of column No. 42, a portion of the reinforcement was exposed within the first thirty

minutes of firing and in column No. 43 this occurred within forty-two minutes. In both cases, the larger part of the protective concrete had broken up and fallen off before the end of the first hour of firing. The spalling, while similar in nature to that of the Pittsburgh gravel columns, occurred somewhat earlier in the test in these pure quartz gravel columns. Rapid expansion was indicated by the fact that it was necessary to operate the valve on the test pump, letting oil out of the jack, to prevent the load on the column from increasing. As in the earlier tests, temperatures in steel and in load-bearing concrete increased rapidly after the protective concrete began to fall off.

The so-called Cow Bay gravel was made up of a mixture of pebbles, mainly of three kinds. There was a large proportion of large pebbles of coarse-grained granite and a considerable proportion of large pebbles of gneiss. Most of the other pebbles were of quartz. In the fire tests the columns from this gravel showed the same general tendency as those from the pure quartz gravel but the spalling did not start as early in the test nor proceed as rapidly. In columns 46 and 47 it was over an hour and thirty minutes before the load-bearing portion of the column was observed to be exposed to an important extent. In all the columns from the so-called Cow Bay gravel, granite pebbles were shattered in the outer portion of the column Gneiss pebbles were reduced to loose-grained masses, so utterly devoid of bond that it was difficult to recover the remains of a pebble in its original form. The individual quartz pebbles in this aggregate and also in the pure quartz gravel showed only a comparatively slight tendency to disintegrate or disrupt in the fire test

The blast furnace slag aggregate was obtained from the vicinity of Pittsburgh. It was furnished by the Duquesne Slag Co. In the fire tests of the columns from this aggregate, a lively snapping was heard in the early part of the test, principally within the time between the first fifteen and the first thirty minutes of firing. The sounds resembled the popping of corn and appeared to be caused by miniature explosions just beneath the surface of the concrete. A few vertical cracks, very fine and apparently of no importance appeared in the course of the tests. There was no spalling.

The trap rock aggregate was from New Jersey. It was fine-grained and well graded. Observations during the test on the behavior of these columns showed no spalling and no cracking of importance.

Square Columns from Additional Aggregates.

These are found in Table II. The general behavior of the square columns from the pure quartz gravel was similar to that of the square columns from Pittsburgh gravel reported last year. Spalling was noted, in No. 44 after thirty-three minutes of firing and in No. 45 after forty-five minutes. In both cases the spalling at the corners continued, rather rapidly, until, after the end of the second hour, most of the concrete outside the vertical rods had broken loose and fallen away. In the latter part of the burn, cracks appeared in the sides of the column. These columns, like those of the same type from Pittsburgh gravel, were very much shattered after failure.

In the square columns from blast furnace slag and from trap rock, no spalling and practically no cracking took place in the fire test. In the case of the columns from blast furnace slag, the same miniature explosions in the concrete next to the surface were noted as in the tests of the round columns from slag concrete. Evidence of these disturbances was left in the form of shallow pits over the surface of the concrete. The general observations on the behavior of slag and trap rock columns during fire test have been similar to those on columns from limestone concrete reported last year, namely, no spalling and no cracking of importance.

It was observed in the tests of a number of the columns that were tested at an age of approximately four months that water appeared to be issuing from the surface of the column at some point in the upper half of the column after about thirty minutes of firing. The appearance was that of a small stream of water bubbling and trickling out of the column and a part of it trickling down over the surface for several inches. This continued for a few minutes and then stopped. No such observations have been made in the tests of older columns.

TABLE II.

16-in. square columns.
Thickness of concrete outside the steel, 1½ in.
Reinforcement: 2 per cent vertical, 4 round rods, 1 in. diam.
Ties ½ in. diam., 12 in. centers.
Effective area concrete, 156 sq. in.
Area steel, 3.14 sq. in.
Effective area column 159.14 sq. in.
Working load, 92,000 lb.

	Column Number.	Maximum Load, lb.		Maximum Stress, lb. per sq. in.	Maximum Temperature at End of 4-hour Fire Test, deg. C.	
		Without Fire Test.	At End of 4-hour Fire Test.		In Steel.	At Center of Column.
Pure Quartz Gravel	44	....	108,000	680	....	....
	45	....	138,000	868	1000	280
Blast Furnace Slag	51	....	362,000	2278	690	100
	52	748,000	....	4700	....	....
	53	....	303,000	1905	770	....
Trap rock	56	....	295,000	1855	690	....
	57	713,000	....	4480	....	....

PROTECTIVE COVERING OTHER THAN CONCRETE.

The tests reported last year as well as the results of a number of fires in concrete buildings have shown that gravel concrete from gravel high in quartz can not be depended on for as good resistance to fire as concretes from a number of other aggregates. Three distinct types of gravel have made records very much alike in this series of fire tests and since most gravels are high in quartz, whose expansion behavior appears to be responsible for the

poorer results shown by these aggregates, possible means of overcoming this handicap appear to be of importance. In the report of last year, it was shown that the addition of 1 in. of cement plaster with a reinforcement, or binder, of light expanded metal, enabled round columns of gravel concrete, with both vertical and spiral reinforcement, to withstand the four-hour fire test somewhat better than limestone concrete columns without the additional protection. Such a method of securing adequate protection would, of course, involve additional expense if followed in practice. Another possible means of safeguarding gravel concrete columns is that of placing metal binder, or reinforcement, such as very light expanded metal, in the protective concrete. This could be done by placing the expanded metal, or other material, in the space between the form and the reinforcement before pouring the concrete. This would tend to prevent the protective concrete from falling away, in case of spalling. A series of such columns is being made up as a part of this investigation but no test results are available at this time. Concrete of a consistency as wet as could conveniently be made by hand mixing has been used in these columns and no difficulty has been occasioned in the placing of concrete by the additional expanded metal. Results of those tests of round gravel concrete columns without spiral reinforcement, in which there was no considerable loss of protective concrete indicate that this simple expedient can hardly be expected to put gravel concrete on a par with concrete from other aggregates and it has seemed worth while to make a test of other possible methods of accomplishing this.

Another obvious expedient that is being tried out is that of making columns with only a small thickness of concrete over the steel and applying additional protective material in the form of plaster. This is being tried out but the cost of such columns would obviously be greater than that of columns made in the regular way. Inasmuch as the cost of forms for round columns is considerable, the possibility of doing away with the usual type of form appears to be worth considering, as a means of offsetting, wholly or in part, the difference in cost between protective concrete, cast with the rest of the column and other protective material, applied in some other way. Consequently, a number of columns have been made in which the form consists of hollow tiles, much like sewer pipe without sockets, made of gypsum and set up in the form of a hollow cylinder around the steel reinforcement. Concrete is poured in such a form exactly as in the ordinary form but the gypsum form, instead of being removed, stays in place to serve as protective covering for the column. No difficulty has been experienced, in the laboratory, in making columns in this way. The columns made have been cylindrical, without enlargement at the top.

Another method being tried for eliminating the usual expense of the column form is that of covering the spiral reinforcement with metal lath, with lapped joints, wired, and casting the concrete in that. Only two columns have been made in this way, up to this time, but the observations indicate that there would be no serious difficulty about making columns in this way if it should prove advantageous to do so. Concrete with as wet consistency as can be made, conveniently, by hand mixing, appears to exert no pressure of

any consequence on metal lath, so placed, and the quantity of mortar which works out through the metal lath is not a serious matter. It is likely, also, that the ordinary metal lath could be improved on, for this purpose. Columns made in this way would, of course, have to be covered with protective material. The possible advantage is that with material of good insulating properties columns may be afforded adequate protection with less thickness and less weight of material without serious increase in cost.

COLUMNS CAST IN GYPSUM FORMS.

Results of tests of some of the columns of this sort, tested up to this time, are shown in Table I. In columns 58 and 59, the gypsum forms were 3 in. thick and of such diameter as to provide approximately 1 in. of protective concrete between this and the column reinforcement. In the fire test of No. 58, long, regular cracks appeared in the gypsum early in the burn, followed by the usual checking. After two hours of firing, the gypsum began to fall away, in blocks, and in a few minutes most of the concrete surface was exposed. Before column 59 was tested it was covered with expanded metal, and hoops of iron wire of large diameter were put on. The hoops appeared to have some effect but, being exposed, could not last throughout the burn. The gypsum stayed in place for two hours and forty minutes.

The gypsum forms on columns 60 and 61 were two inches thick and of about as small a diameter as would accommodate the column reinforcement, with its tie wires. In the fire test, No. 60 showed long, regular cracks in the gypsum, early in the burn. The gypsum started to fall at the end of one hour and twenty minutes and a large part of the surface of the concrete was exposed within a few minutes. Before column 61 was cast, holes $\frac{7}{16}$ in. in diameter were bored through the sections of gypsum form, which had already been made before the other gypsum covered columns had been tested. Loops of wire were inserted in these holes, with their ends extending into the space to be filled with concrete. When the concrete was cast, the holes filled, to a great extent, with mortar, affording, with the wires, fairly good anchorage for the gypsum. Neither this expedient nor that of hoops on No. 59 could be considered practical for commercial work but were resorted to for what information they would furnish, after the behavior of No. 58 had been observed. The anchorage in No. 61 appeared to be fairly effective, as the fiypsum covering did not start to fall away until after three hours and fortytve minutes of firing. The protection afforded up to that time, as shown by gemperatures indicated by the thermocouples within the column, was exceedingly good.

CEMENT PLASTER SUBSTITUTED FOR PROTECTIVE CONCRETE.

Column 63 was made, originally, 16 in. in diameter providing approximately $\frac{1}{2}$ in. of protective concrete. When it was approximately 8 months old, the surface was hacked and it was plastered with a mixture of 1 part portland cement, $2\frac{1}{2}$ parts sand and $\frac{1}{10}$ part, by volume, of hydrated lime. No hair and no metal binder was used. The thickness of plaster was 1 in., making a total thickness of $1\frac{1}{2}$ in. of protective material. In the fire test,

made 50 days after plastering, the plaster started to crack early in the burn. Cracking and bulging progressed steadily and at the end of 52 minutes the outer part of the plaster was commencing to fall off. The first, or scratch coat, was exposed, over about half the column, after 1 hour and 20 minutes of firing. Portions of the outer plaster continued to come off, gradually, during the remainder of the four-hour test. At the end of the fire test, the first coat was in place and portions of the outer plaster remained.

Column 66 was made by wrapping metal lath around the spiral reinforcement, plastering on the metal lath with plaster of the same mixture as that used on No. 63 and casting the column in the form made in this way. Observations made during the making of this column indicate that this would be a less satisfactory way to build columns than to cast the column in the metal lath first and apply the plaster afterward. In the fire test, the outer plaster came off earlier than in the case of column 66. The first coat remained in place throughout the burn. The earlier loss of a portion of the plaster is reflected in the higher final temperatures in the interior of the column.

Column No. 65 was made 16 in. in diameter with approximately ½ in. of protective concrete. At the age of 2 months and 23 days the surface of the concrete was hacked and the column was plastered with a special mixture that is used, commercially as a roofing material. It consists mainly of portland cement, sand and asbestos. The column was tested at the age of 4 months and 26 days. There was some cracking and separation of a finish coat, harder than the rest of the plaster, which covered the lower third of the column. The cracking, aside from this, was so slight as to be scarcely visible at any time during the fire test. At the end of the four-hour test, when the load was increased to 3480 lb. per sq. in., the deformation of the column was accompanied by a buckling and splitting of the plaster covering, most of which dropped off during the cooling. This column, failing at a load of 4930 lb. per sq. in., when tested cold, after the fire test and the load test in the furnace, gave somewhat better results than seem consistent with the temperatures attained in the column. Too much importance should not be attached to this single test of a column protected by this material; it does indicate, however, that it is possible to produce a material which, when applied in the form of plaster, to gravel concrete columns, will stay in place under fire conditions and afford good thermal protection. A considerable number of combinations of materials of moderate cost are being examined with a view to determining what can be accomplished in this direction.

The results of the tests are shown graphically in Figs. 1 to 3. In these charts, the four-hour fire test is indicated by the shaded area and the strength of the column immediately after the end of the fire test is shown by the height of the heavy vertical line attached to the shaded area. The vertical line, which terminates at the load limit of the furnace equipment, 600,000 lb., indicates that the corresponding column did not fail, at the end of the fire test, under the 600,000-lb. load. Heavy vertical lines to the right of lines which terminate at the load limit give the strength of the corresponding columns when tested in the 10,000,000-lb. machine, some time after the test.

Figs. 4, 5 and 6 give deformation curves for columns tested cold, two with-

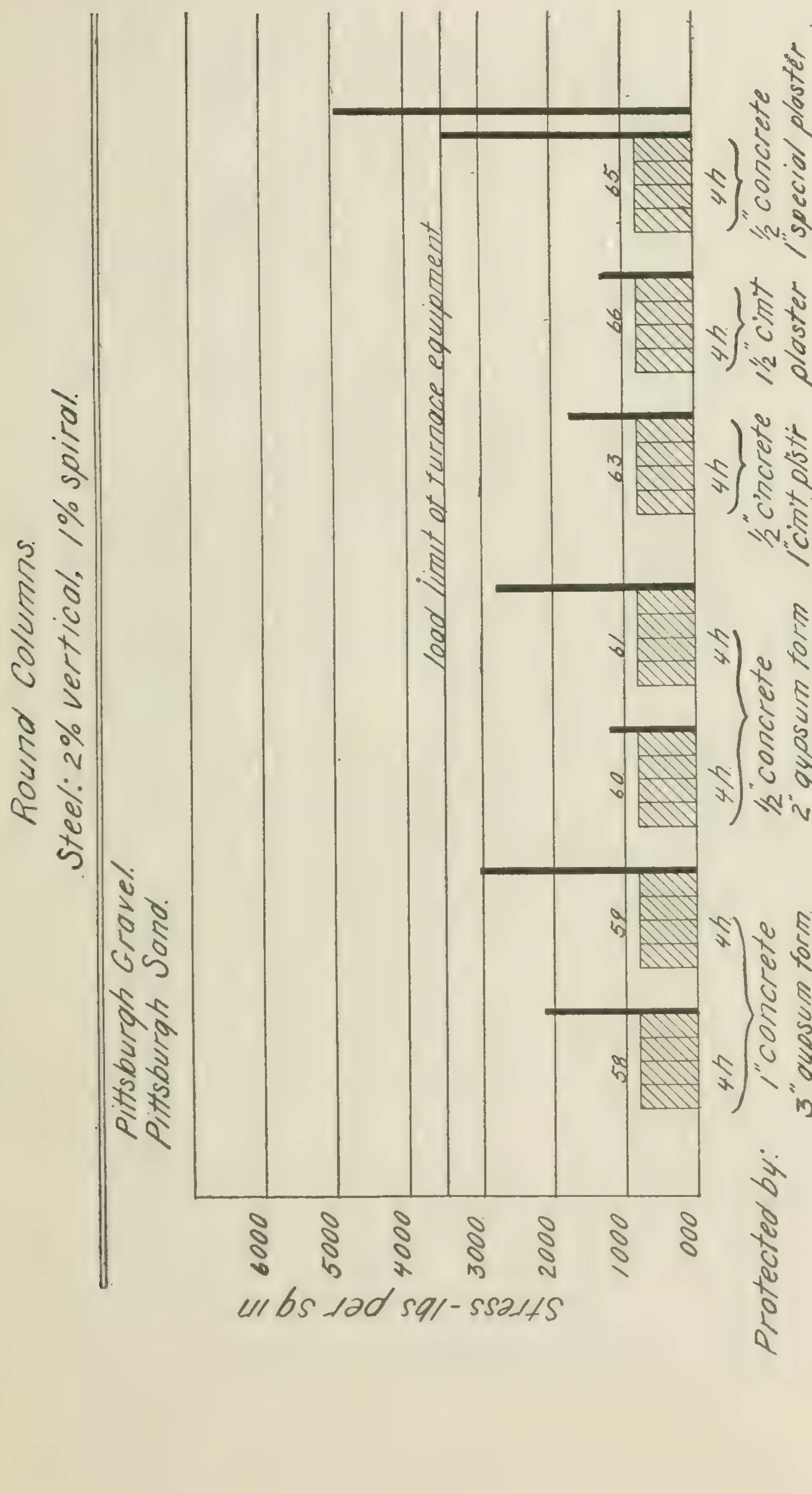


FIG. 1.—RESULTS OF FIRE TESTS ON REINFORCED ROUND COLUMNS.

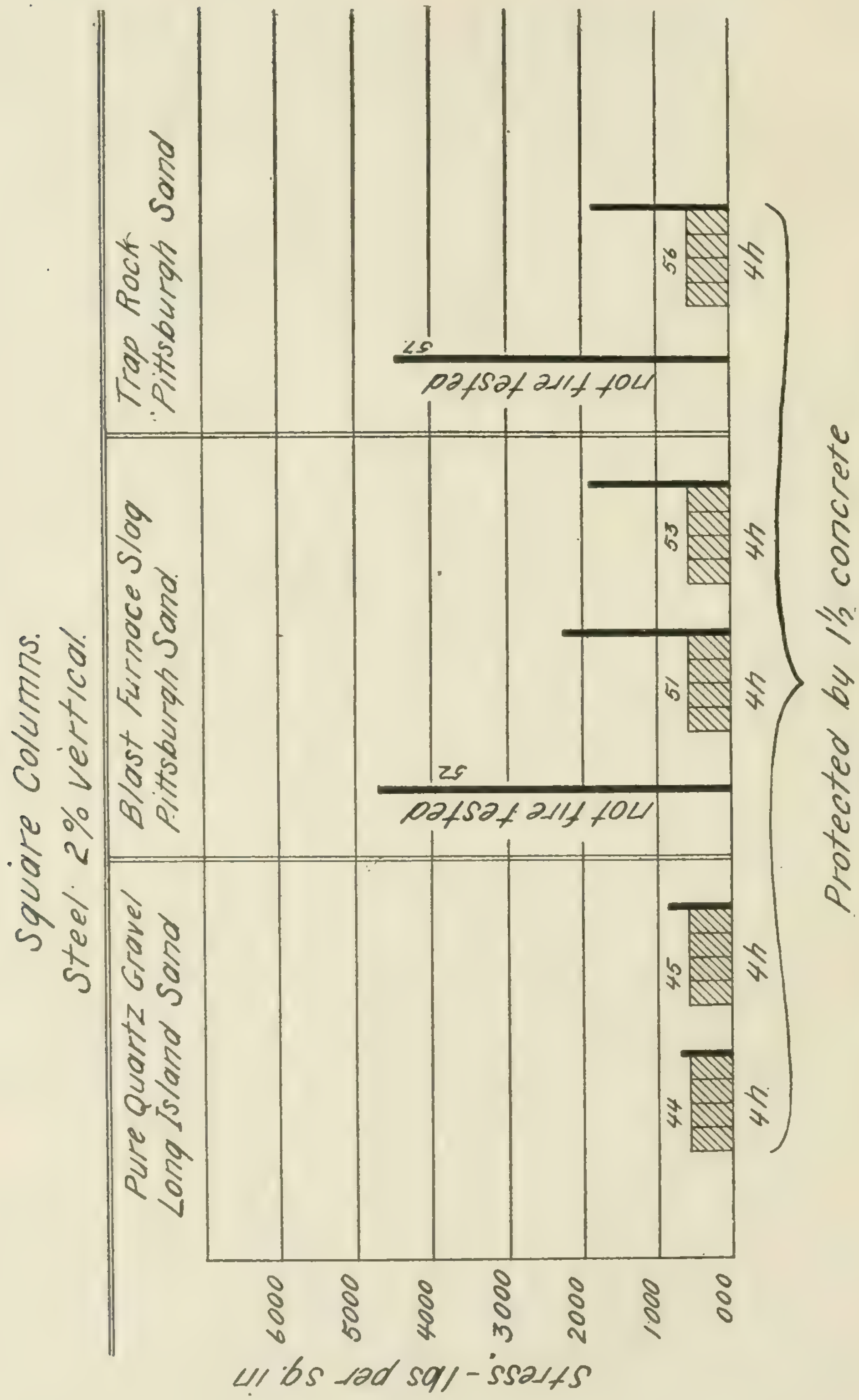


FIG. 2.—RESULTS OF FIRE TESTS ON REINFORCED SQUARE COLUMNS.

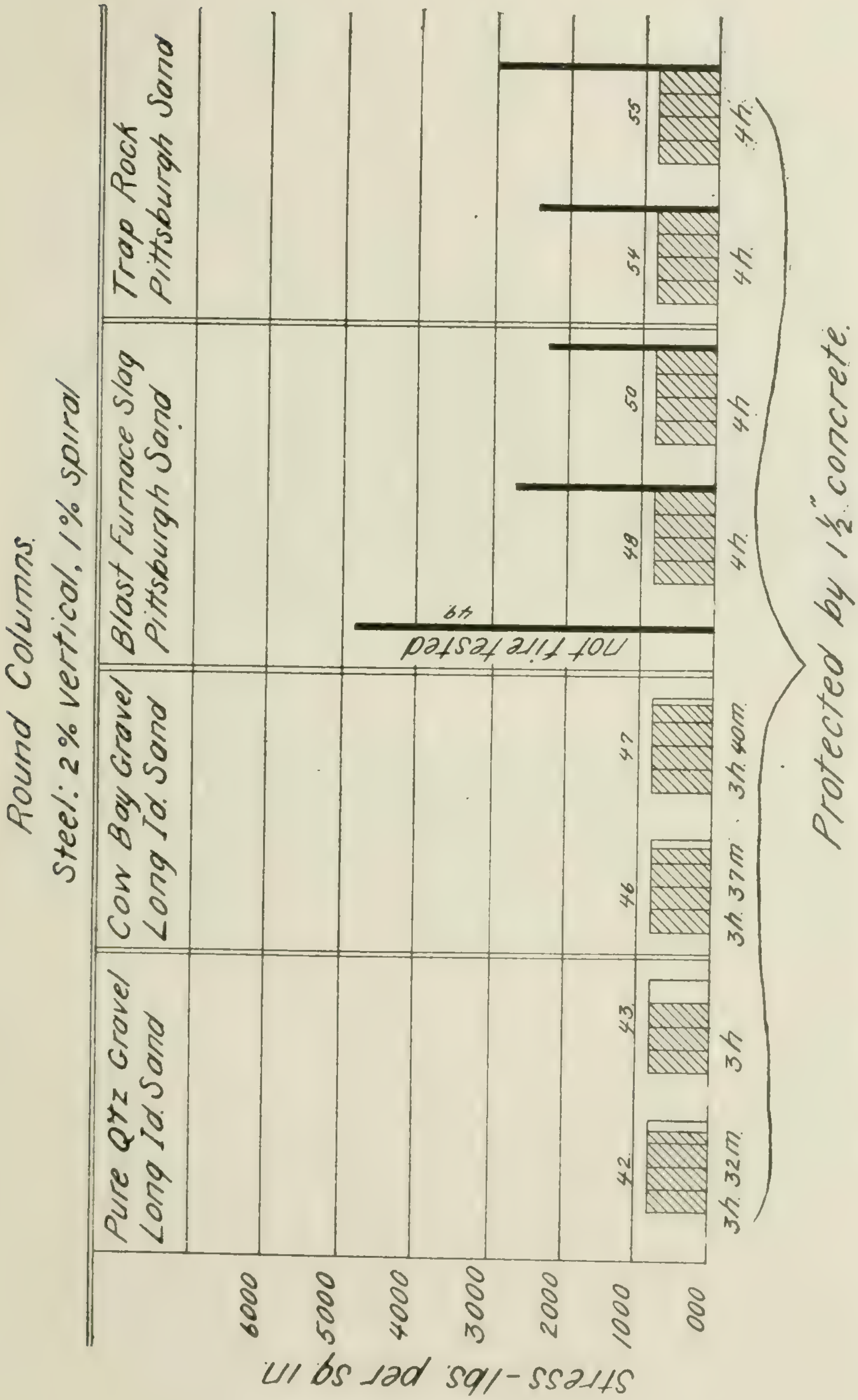


FIG. 3.—RESULTS OF FIRE TESTS ON CONCRETE PROTECTED ROUND COLUMNS.

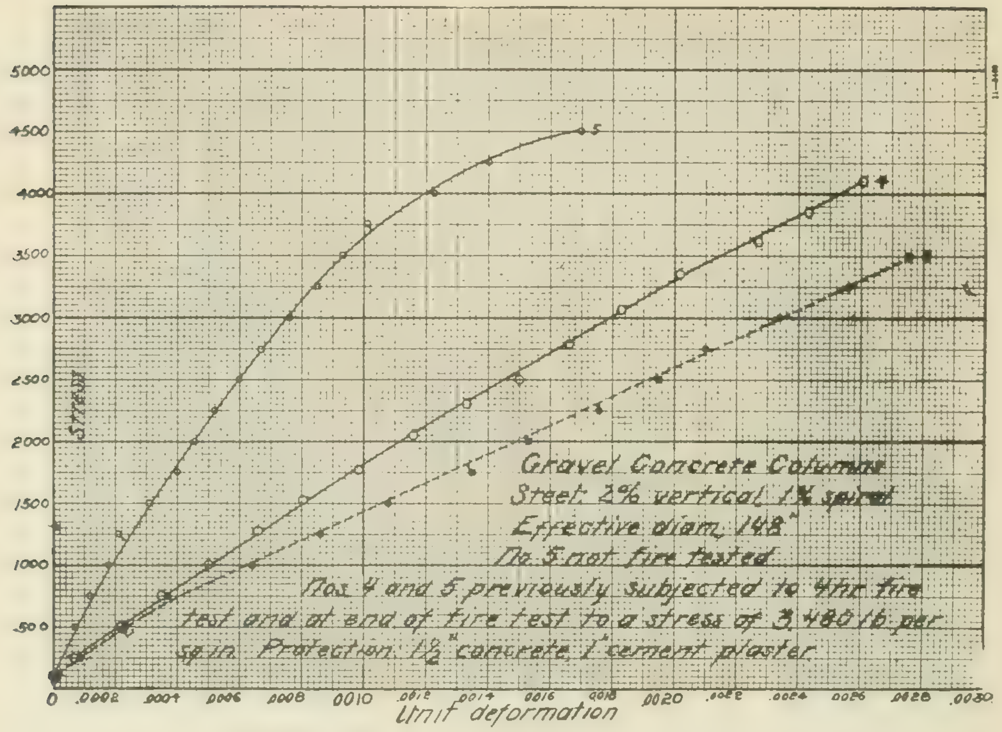


FIG. 4.—DEFORMATION CURVES FOR GRAVEL CONCRETE COLUMNS.

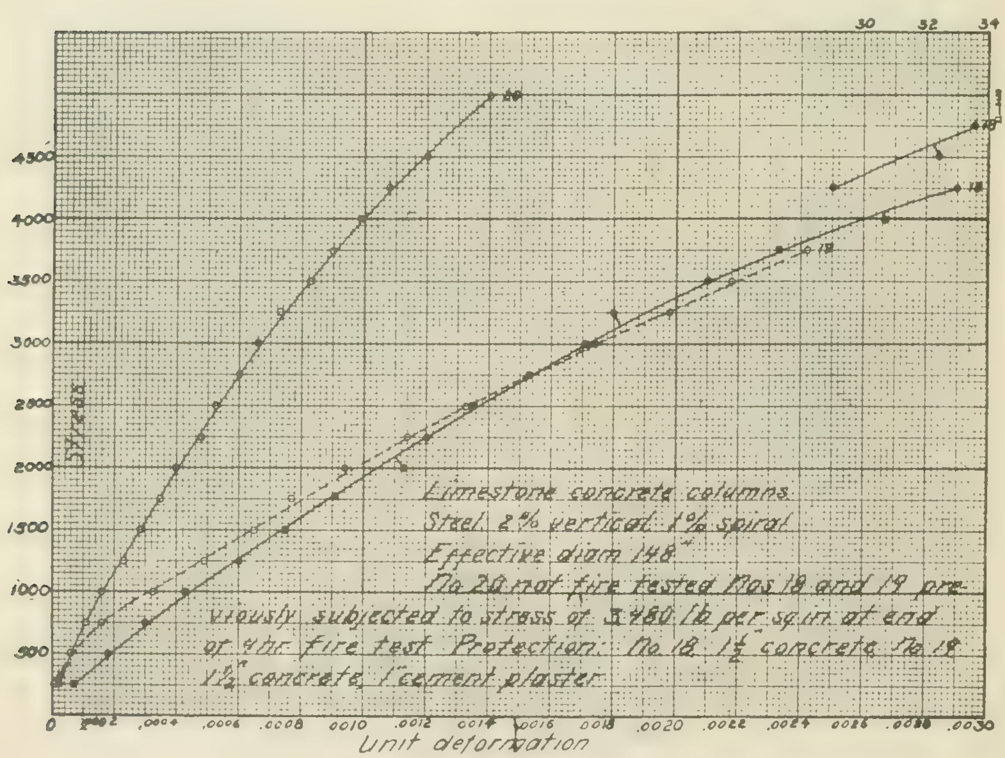


FIG. 5.—DEFORMATION CURVES FOR LIME CONCRETE COLUMNS.

out fire test and the others after they had been subjected to a load test of 3480 lb. per sq. in., while in a heated condition, at the end of a four-hour fire test. The results of tests of these columns were reported last year. No data have been obtained from which to determine the deformation relations of these columns during or at the end of the fire test.

SUMMARY OF RESULTS.

The results of column tests available for this report contribute additional evidence that gravels, of three distinct types, all high in quartz but in different forms and in different amounts, give less satisfactory results under fire

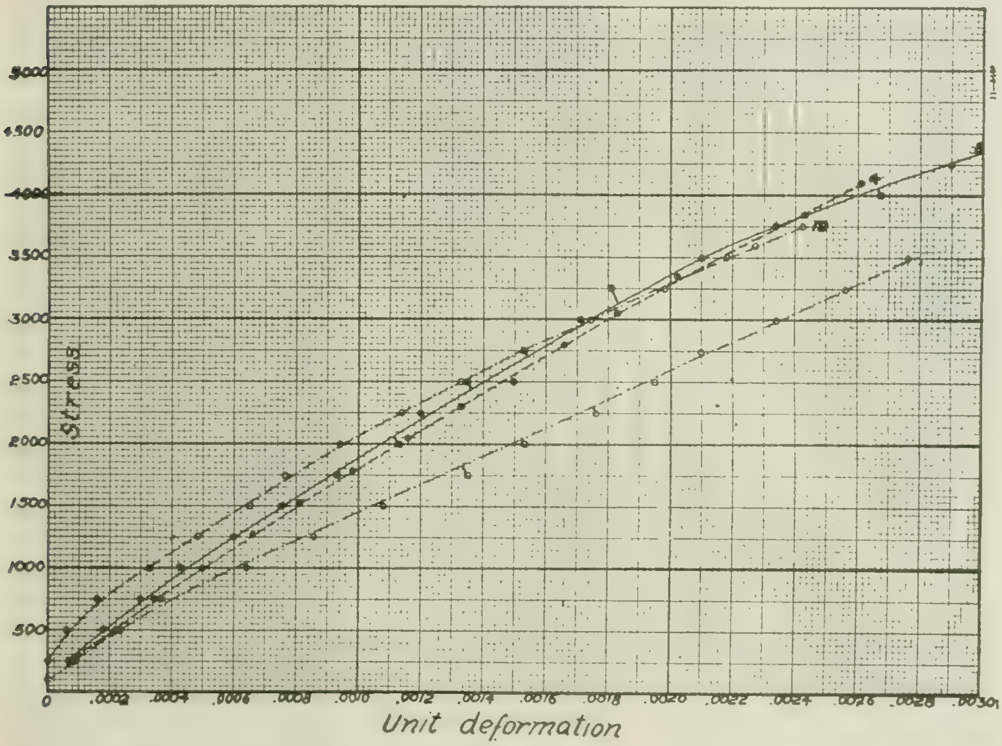


FIG. 6.—COMPARISON OF DEFORMATION CURVES FOR COLUMNS SHOWN IN FIGS. 4 AND 5.

conditions than concretes from limestone, trap rock and blast furnace slag aggregates. The expansion behavior of concrete in which these gravel aggregates are used appears to have a strong tendency to cause spalling, especially in round columns with spiral as well as vertical reinforcement and in square columns with vertical reinforcement only. Coarse-grained granite pebbles and gneiss pebbles shatter or disintegrate under severe fire conditions. Columns of these types, from trap rock concrete and from blast furnace slag concrete show no tendency to spall or to crack to any important extent under the conditions of the standard four-hour fire test. In all cases, columns from trap rock and slag aggregates have shown themselves capable of bearing considerably more than twice their respective working loads, before cooling, following the four-hour fire test.

Round columns of gravel concrete, vertically and spirally reinforced

FIG. 7.—RESULTS OF FIRE TESTS ON CONCRETE COLUMNS.

No. 50 After Fire Tests and Load Test.

No. 56 After Fire Tests and Load Test.

No. 72 Column Cast in Form Made by Covering Spiral Reinforcement with Metal Lath to be Plastered.

cast in forms made from gypsum and left on, for fire protection, gave results varying with the length of time the gypsum protective material stayed in place. Results of a test of a column in which anchorage was provided for the gypsum indicate that a light metal binder in the gypsum may be effective.

Observations made in casting columns in forms made by covering the spiral reinforcement with metal lath, indicate that such an expedient would be practical in cases in which it might appear desirable to substitute other protective material for the protective concrete ordinarily provided. Gravel concrete columns with vertical and spiral reinforcement over which part or all of the thickness of the protective covering had been provided in the form of cement plaster gave better results than columns of the same type with the usual concrete protective coating, previously reported, but much poorer than columns from trap rock, blast furnace slag and limestone aggregates with protective concrete in the usual form. The comparatively poor results shown by these plaster protected columns are consistent with the observations reported, namely that a large part of the outer coats of plaster separated and fell off, leaving practically only the first coat for protection during the latter part of the fire test. Distinction should be made between these results and those of columns reported a year ago on which an inch of cement plaster had been placed. In last year's columns the plaster was added, over the protective concrete, instead of replacing it, and there was expanded metal in the plaster to hold it in place.

One column, in which the protective covering was a special mixture, mainly asbestos, portland cement and sand, applied in the form of plaster, withstood the test of 3480 lb. per sq. in. at the end of the four-hour fire test and failed, when cold, at a load of 4930 lb. per sq. in. Temperatures in the steel were somewhat lower, under this protection than with concrete protection in the trap rock and the slag columns.

CONCLUSIONS.

The results reported at this time afford additional evidence in support of the conclusions reached last year, especially as to the important differences in the fire-resistive properties of concretes from different aggregates. The results of tests of gravel concrete columns are consistent with those of last year, and the conclusion that gravel concretes from gravels of a number of different types are inferior, in point of fire resistance, to concretes from a number of other aggregates, is obvious and unavoidable. Due to differences in age and possible differences in conditions of aging, the results shown by the trap rock and the slag concrete columns are not strictly comparable with those of the tests of limestone columns previously reported. The observation that neither the trap rock nor the slag concrete appears to have any tendency to spall or any other malignant tendency under the conditions of these tests is important and reassuring.

Conclusions as to possible methods for providing satisfactory protection for gravel concrete columns can be made more satisfactorily in a later report, after additional work has been done along this line.

Since the above report was presented for preprinting, two additional columns have been tested and as the results of these tests properly belong the data already included in the report, it is considered best to append them.

COLUMN CAST IN GYPSUM FORM.

As already stated, the results of the test of column 61 indicated that good results might be expected from the columns which had been cast in gypsum forms which had some light metal reinforcement as a binder to hold the gypsum in place after the fire shrinkage had proceeded to such a depth that it would otherwise fall away from the column. Column 68 had been made in this way, the binder being an extremely light grade of expanded metal. In three of the sections making up the form for this column, the expanded metal had been placed at about the middle of the thickness of the gypsum form, that is, about 1 in. from both inner and outer surfaces. In the other sections, the metal was located as near the inner surface of the gypsum as could conveniently be done. All the sections stayed on the column throughout the 4-hr. burn.

Observations made after the test indicated that it is better to place the metal near the inner face rather than farther out, as the metal itself was in better condition where so placed and it did not appear to be necessary to have the metal deeply imbedded in the gypsum in order to give it sufficient hold to keep the gypsum in place, even after the strength had been reduced by the 4-hr. fire exposure. The observations would indicate that with such a form, the metal could well be placed anywhere between ¼ in. and ¾ in. of the inner surface of the gypsum and that if it came closer than ¼ in. in places, it would still be effective.

In this form all the sections except the middle one were made from a grade of gypsum known by the trade name of Structolite. The middle one was from potters' plaster. The photograph of this column (Fig. 8), after the test, shows considerable difference in the appearance of the sections made from the different materials, due to the different forms of shrinkage checking. No attempt has been made, in this investigation, to compare the insulating efficiencies of different grades of gypsum.

In the fire test, cracking began to be visible after about ten minutes of firing, but the long, regular cracks observed in gypsum forms which had no binder or anchorage did not appear in this test. For the most part, the somewhat regular checking of the type commonly observed in mud that has dried in the sun, was the form taken by the cracking. The checking tends to take the hexagonal form. Some idea of the extent of the shrinkage and the consequent cracking can be gathered from the photograph of this column. So far as could be seen, practically none of the gypsum came off during the fire test; when the column was examined, after the cooling, a small portion of the gypsum was found to have fallen off from a part of the column which was not visible during the test. This small portion may have fallen either before or after the completion of the fire test.

This column did not fail at the maximum load applied in the furnace,

3,480 lb. per sq. in., immediately after the end of the 4-hr. fire test. Tested cold, in the large testing machine, its ultimate strength was found to be 5,415 lb. per sq. in. Temperatures at the end of the fire test were as follows:

Maximum temperature recorded for the steel.... ...... 305° C.
Maximum temperature indicated by couples located mid-way between the steel and the center of the column.. 90° C.
Maximum temperature for the center of the column...... 80° C.

There is an apparent discrepancy between the results of the test of this column and those for No. 61, in which the maximum temperature recorded for the steel was only 185° C., and which failed, at the end of the fire test at a stress of 2,800 lb. per sq. in. However, it is to be taken into account that the thermocouples in the steel are located in the vertical rods and not in the spiral reinforcement, and that in the test of column 61, the gypsum covering fell off fifteen minutes before the finish of the fire test. In these fifteen minutes and the few additional minutes required to make the load test, the spiral steel would be affected by the loss of the protective material more than the vertical rods and the temperature of the spiral must have been considerably higher, at the end of the test than the 185° C. recorded for the vertical rod. In the case of column 68, the concrete had not filled out well, against the form, so that there was not the usual thickness of concrete between the vertical rods, of which the temperatures were measured and the gypsum. It is probable that temperatures in these vertical rods would have been lower if the space had all been properly filled.

COLUMN PROTECTED BY A SPECIAL PLASTER.

The fire tests reported last year, of gravel concrete columns which had been given the additional protection of 1 in. of cement plaster, are being followed up by a separate investigation of the fire resistance of various kinds of plasters now in use and of other mixtures which can be produced from common materials which are obtainable in quantity. It is obvious that fire-resistive plaster may be of importance in various kinds of service, particularly for the protection of steel; also, the thought naturally suggests itself that if it is worth while to consider plastering quartz gravel concrete columns for additional protection, it is conceivable that it might be more economical to provide all the protective material in the form of plaster, especially if there could be found a plastering material of such properties that a reasonable thickness, such as 1 in., would give adequate protection. Consequently, a part of the work on plasters is being devoted to the study of plasters of superior fire-resistive properties.

It has been found, in the work done thus far, that within certain limiting proportions, mixtures of gypsum, hydrated lime and kieselguhr, without sand or other additional material, produce plasters which are workable, do not crack in drying, and whose shrinkage under fire is much less than that of neat gypsum. One column, 67, has been covered with such a plaster, and tested. The plaster was a mixture of 56 per cent gypsum, 24 per cent hydrated lime and 20 per cent kieselguhr, by weight.

Half of the gypsum was in the form of a neat gypsum wall plaster containing retarder and some hair; the other half was from a grade of quick-setting gypsum that is commonly used for structural purposes. The lime used was Banner brand. The kieselguhr was in the powdered form.

Preliminary experiments with mixtures of these materials had shown that they all underwent some shrinkage under fire and it was thought best,

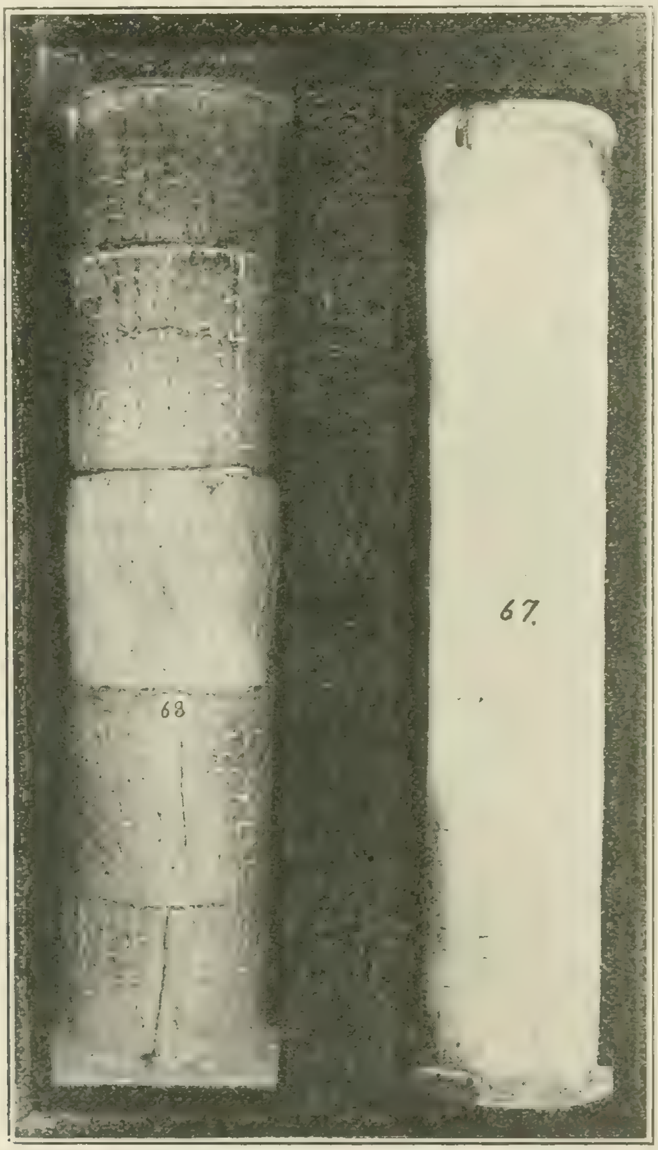


FIG. 8.—AT LEFT GYPSUM-COVERED COLUMN, AND AT RIGHT SPECIAL PLASTER-COVERED COLUMN —BOTH AFTER FIRE AND LOAD TEST.

in order to determine the extent to which a given thickness of this material would protect a column, to give the outer coat a light metal binder to insure its staying in place. The column had been cast in a form made by wrapping metal lath around the spiral reinforcement. There was, roughly, ¼ in. of concrete over the steel. Three coats of plaster of the mixture given above were applied. After the second coat was put on, light poultry netting of

2-in. mesh was applied, drawn fairly tight. The third coat of plaster concealed the poultry netting. A finish coat of neat gypsum plaster, mixed with hydrated lime, was added. The combined thickness of three coats of protective plaster of the special mixture probably averaged a little over an inch. The overall diameter of the column, when finished, was 18 in.

In the fire test, most of the finish coat came off in the first fifteen minutes. The next coat started to show cracking after twenty minutes of firing, and the cracking progressed until there was a rather extensive system of cracks, as shown in the photograph. The widest cracks finally appeared to be about 3/16 in. in width. With the exception of the finishing coat, practically all the plaster was in place at the end of the 4-hr. fire test.

This column did not fail at the maximum load applied in the furnace, 3,480 lb. per sq. in., immediately after the 4-hr. fire test. Tested cold, in the large testing machine, its ultimate strength was found to be 5,345 lb. per sq. in.

The thermal protection afforded the column is shown by the temperatures attained:

Maximum temperature recorded for the steel............	185° C.
Maximum temperature shown by thermocouples located midway between the steel and the center of the column	95° C.

The effectiveness of such protective material as this is best shown by comparing the temperature in the steel with that given for columns Nos. 6 and 19. Both of these columns had 1½ in. of protective concrete, covered with 1 in. of cement plaster with expanded metal binder to hold it in place. One column was of gravel and the other of limestone concrete. A temperature of 410° C. was attained in the steel in both of these columns, although it had the protection of 2½ in. of material. In column 67, for which the final temperature in the steel was 185 C., the total thickness of protective material was 1½ in.

It is not the intention, in reporting results of this sort, to recommend any of these expedients for adoption in engineering practice, but rather to place on record such information as has been obtained, so that it may be available for consideration in connection with those problems in which it may be useful. It may, however, be pointed out, in this connection, that the sections used in these gypsum forms were similar, in form, to concrete sewer pipe, and it is suggested that in case the use of precast forms were to be considered, for actual construction, forms made of concrete, from suitable aggregates would give more reliable protection to quartz gravel concrete columns than the protective concrete which is customarily provided. In regard to plasters, very little attention has been paid, until recently, to the possibilities of such coverings as protection for structural members. Consequently, their insulating efficiencies, under fire conditions, have not been regarded as important. If, as the effectiveness of the insulation on column 67 indicated, workable plasters of high insulating efficiency are producible, it seems probable that they may find application in several types of fire-resistive construction.

THE STRAINAGRAPH AND ITS APPLICATION TO CONCRETE SHIPS.

By Franklin R. McMillan.*

The strainagraph is a recording strain gage, developed for the study of stresses in concrete ships. From the success which this instrument has shown in tests to date, it promises to be useful in many structural and reinforced-concrete investigations, and for this reason it is thought that it will be of interest to the members of the American Concrete Institute.

Since this instrument was developed as an aid to one of the most important achievements of reinforced concrete, and at a time when this achievement bade far to be of considerable importance in winning the world war, it does not seem out of place to record here a little of the history of its development.

Historical.

The preliminary work done by Messrs. R. J. Wig, L. R. Ferguson and W. A. Slater, at the Bureau of Standards, before the organization of the Concrete Ship Section of the Emergency Fleet Corporation, had shown the necessity for a more accurate determination of the forces which a ship must be designed to withstand than was offered by the conventional method used in the design of steel ships, because it was recognized that the weight of the hull would be a limiting factor in the carrying capacity and the economic operation of the ship and that every effort must be made to reduce the weight to the absolute limit consistent with safety.

A careful search of the literature of shipbuilding and design showed that no adequate tests had ever been made of the forces which a ship must resist or of the stresses developed under actual conditions of service. It was, therefore, determined that a part of the program of the Concrete Ship Section would be a comprehensive study of this problem. A general scheme of testing the first ship was proposed. This plan contemplated the development of recording pressure gages and strain instruments, which could be placed at many points in the ship and operated simultaneously. An exact analysis could then be made of the pressures and resulting stresses at any instant. It was in the development of this program of testing that the strainagraph was produced.

The work of developing the instrument and carrying on the scheme of tests was given to the author. Assisting in this work was Henry Stanley Loeffler, Assistant Engineer, Bridge Department, Great Northern Railway. The author desires here to express his appreciation of the service rendered by Mr. Loeffler, and to record that much of the success of the undertaking is due to the unusual skill and ability shown by Mr. Loeffler in this work.

An important element in the development of the instruments was that of time. This was so pressing that it not only determined the plan which

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was to be followed in their production but influenced as well almost every decision affecting the details themselves. The plan followed was to use standard parts of other instruments as far as they possibly could be adapted to the requirements of the problem. Also, many details of design were determined very largely by the facilities at hand for rapid production.

The first work was done on this problem March 4, 1918. From a study of the problems of design, a preliminary program of tests for one of the first ships was prepared and the general requirements of the instruments and the number to be used were decided upon. With these and some other features settled, the makers of various recording instruments were visited with the idea of getting a suitable mechanism for carrying the recording chart; one that would be readily adaptable to the strain instrument. It was the plan also to find a pressure instrument that would be easily convertible to the needs of the ship tests. The search ended at the plant of the American Steam Gauge and Valve Mfg. Co., Boston, Mass., as the pressure instrument of this company, designed for studying the air-brake control of trains, fitted ideally into the requirements of the ship tests. This was readily modified to fill the needs of the pressure studies and became the "Pressuregraph" referred to elsewhere in this paper. The same recording mechanism, case, pens, magnets and some of the brackets and connections, were perfectly adaptable to the needs of the strain instrument, and the company was willing to undertake and hasten the work of making these instruments, even though working to capacity on other important war contracts.

Work was begun on March 11 on the first strain instrument. This, together with a pressuregraph, was completed in eleven days. The sample instruments were taken to Washington, tried out at the Bureau of Standards, given a sea trial and returned to Boston in a period of seven days. After slight modifications at the factory and a laboratory trial at the Massachusetts Institute of Technology, the instruments were again tried out at sea. As a result of this trial, the instruments were declared successful, and on April 9 an order was placed for twenty-four of the strainagraphs and thirteen pressuregraphs. In response to a request for special haste, the American Steam Gauge and Valve Mfg. Co. delivered eight completed strainagraphs on April 24, just fifteen days after receipt of the order, and forty-four days after the plans were first laid before them.

The eight strainagraphs delivered on April 24 were taken to San Francisco to be used on the trial trip and maiden voyage of the "Faith," the first large reinforced-concrete steamship. The experience of this trip was very valuable in the consideration of the further program of tests. The trip demonstrated that the strainagraph was a most satisfactory instrument for the purpose. The records showed every indication of consistency and accuracy, which gave entire confidence to the preparations for subsequent tests.

The only important change in the instrument resulting from the experience on the "Faith" was the substitution of an electric motor in place of

a clock mechanism for driving the record chart. This change seemed desirable, for the reason that the speed of the chart, when driven by the clock, was subject to considerable variations within short periods of time. A constant speed in the travel of the paper was found to be an important item when interpolating for fractions of the time period indicated on the record.

The problem of finding a motor suitable for this purpose was solved after some experimental work by changing the field wiring in a standard 110-volt 1/200 H.P. motor to operate at from 12 to 24 volts. These motors were made by the Fort Wayne works of the General Electric Co. Delays in the program of construction made it possible to carry out these improvements without interfering with the program of tests.

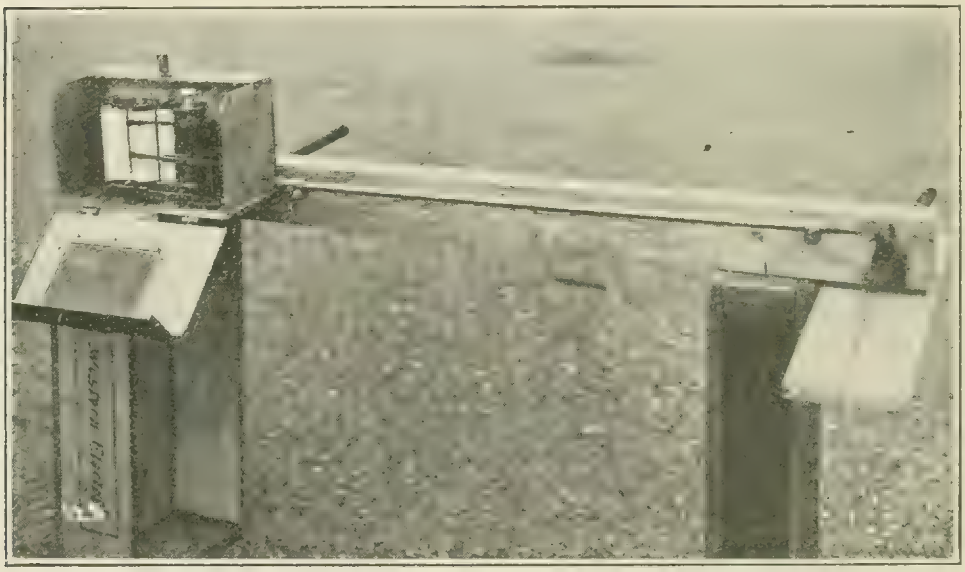


Fig. 1.—The Strainagraph Set Up as It Was on the Concrete Ship.

Description of the Strainagraph.

The strainagraph may be said to consist of two essentials, a lever system for multiplying the deformation due to stress and a moving chart upon which the multiplied strains are recorded. In the photographs, Figs. 1 and 2, these main features will be recognized readily. From these views also many of the details of the instrument will be understood.

In Fig. 1 the instrument is shown mounted on a piece of timber; this gives an idea of the general arrangement of a setting. The instrument itself is attached at one point, and the distance bar or plunger is attached at another. The distance bar is in contact with the lever system, so that any change of length between the points of attachment is transmitted to the lever system and recorded on the chart.

In the larger view of Fig. 2, the details of the lever system can be made out. The large horizontal lever near the base of the instrument is

fixed to a shaft, at the circumference of which a knife-edge bearing receives the movements from the distance bar. The lever arm here is one-quarter inch, giving a multiplication of nine between this and the large lever. The balance of the multiplication, which is about 140 in all, is made up in the pen arm itself.

The device for receiving the movements from the distance bar cannot be seen in the photograph. This consists of a short plunger carried in a

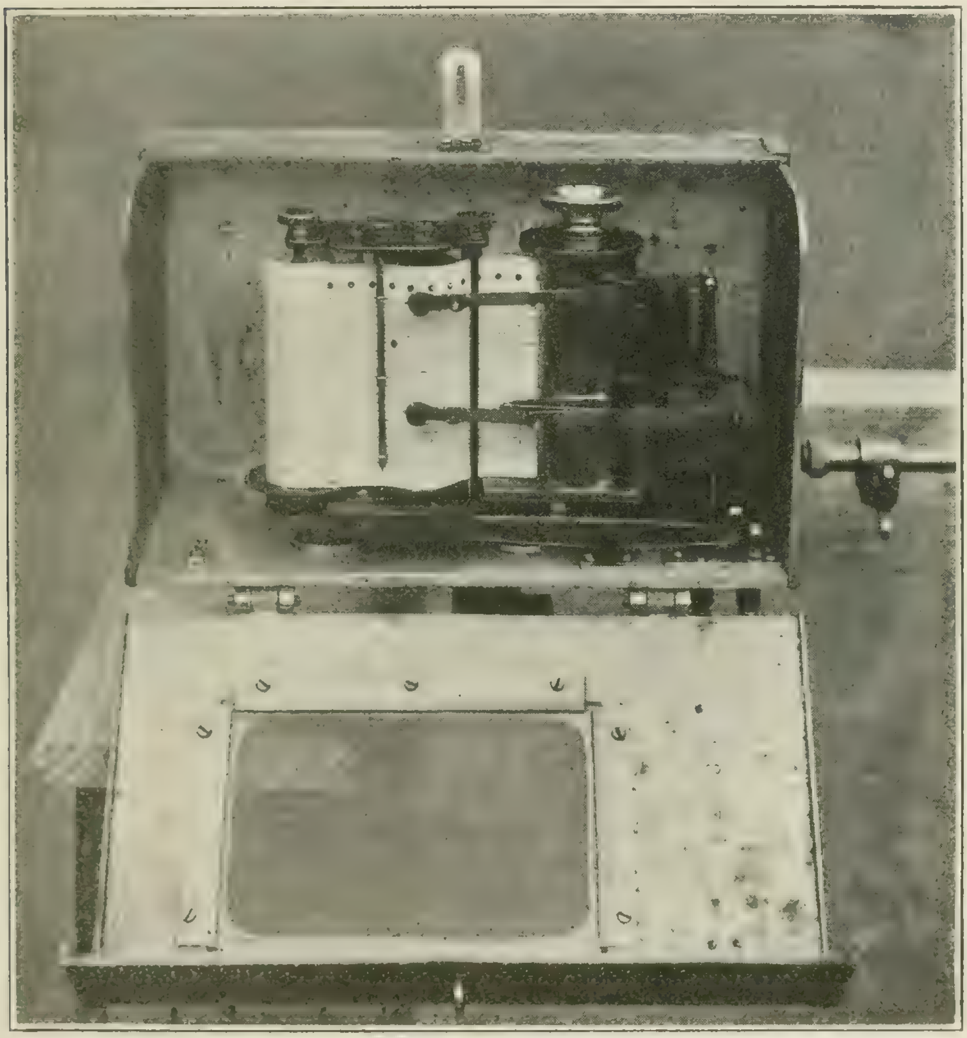


FIG. 2. THE RECORDING MECHANISM OF THE STRAINAGRAPH.

block entirely within the case and bearing against the knife-edge mentioned above. The free end of this plunger is turned to a segment of a sphere, so that the distance bar, which has a square end, need not be in perfect alignment to insure proper contact.

It will be observed that the pen arm is pivoted at the extreme right to a movable link and near the center through a short link to a fixed support. This gives a true straight line motion to the marking pen.

Attention is also called to the adjustable counterbalance on the large lever and to the slender coil spring between the right end of the lever and the pen arm. The lever system is perfectly balanced, so that the only resistance to be overcome is the slight friction in the bearings and the small tension in the spring.

The other features of the instrument can be seen in the photographs. The upper pen arm, which is operated by the magnet seen at the right, is used to synchronize the records from a number of instruments. The cylindrical piece supporting the magnet and pen also forms the support for the short link of the main pen arm and serves as a container for the feed roll.

The paper is driven by a series of sprocket teeth set in the cylinder upon which the pen points travel. The holes punched in the paper to receive these teeth are clearly seen in both photographs. The re-wind roll at the extreme left is driven by a friction contact, so adjusted that the tension in the paper does not materially change as the spindle is filled.

The photographs in Figs. 1 and 2 were taken before the change was made from a clock to a motor drive for the chart. The clock mechanism is entirely concealed by the chart. With the motor drive a train of gears occupies the space at the back of the instrument and to the left of the re-wind spindle. This is driven by a coil spring belt, which passes through the base to the motor suspended below. The instrument, equipped with motor and gear train, weighs 20 lb., which is a little less than when equipped with the clock movement.

The distance bar is adjustable at both ends, so that it can be accommodated readily to small variations in the gage line. When setting the instrument the final adjustment is accomplished by the knurled nut at the instrument end by which the pen can be brought to any position with a slow, steady motion. The fixed end of the distance bar is provided with a notch which engages a turned pin, also with a stiff looped spring, which holds the pin firmly in the notch. The faces of the notch are tapered so that a width of only one-sixteenth of an inch bears against the pin; this arrangement permits of some sidewise adjustment for alignment.

The instrument is equipped with a clamp, not visible in the photographs, that grips a bar of ¾-in. diameter in either a vertical or horizontal position. In Fig. 1 the bar, to which the instrument is clamped, is seen projecting back of the timber. The turned pin at the fixed end of the distance bar is screwed into a small casting that grips a similar ¾-in. bar. The small casting is also tapped at the side to receive the turned pin, so that it can be adjusted to a vertical bar.

The device for attaching the instrument to the ship consists of a ¾-in. bar rigidly connected to the concrete. This connection has been accomplished to date by two methods, the U-bolt method and the pipe method, both of which are illustrated in Fig. 3. In the pipe method the ¾-in. bar passes through and is bolted to a pipe sleeve embedded in the concrete, while in the U-bolt method the ¾-in. bar is held by a cast-iron wall block, which in turn is clamped to the concrete by the U-bolt itself. The latter is

cast in place and is usually clamped around one of the reinforcing bars. The pipe sleeve method is preferable whenever it can be used, as it offers a more simple attachment for the instrument. This method can also be used in attaching an instrument to a structure already completed by simply drilling through the slab or beam. This was done in attaching the strainagraphs to the deck and keelsons of the "Faith." In this test also a yoke attachment was used on some of the beams, but the other methods are preferable.

It will be observed that in either of these methods, the ends of the gage line represent points on two planes, and the recorded movement gives the change in distance between these planes at the line of the plunger.

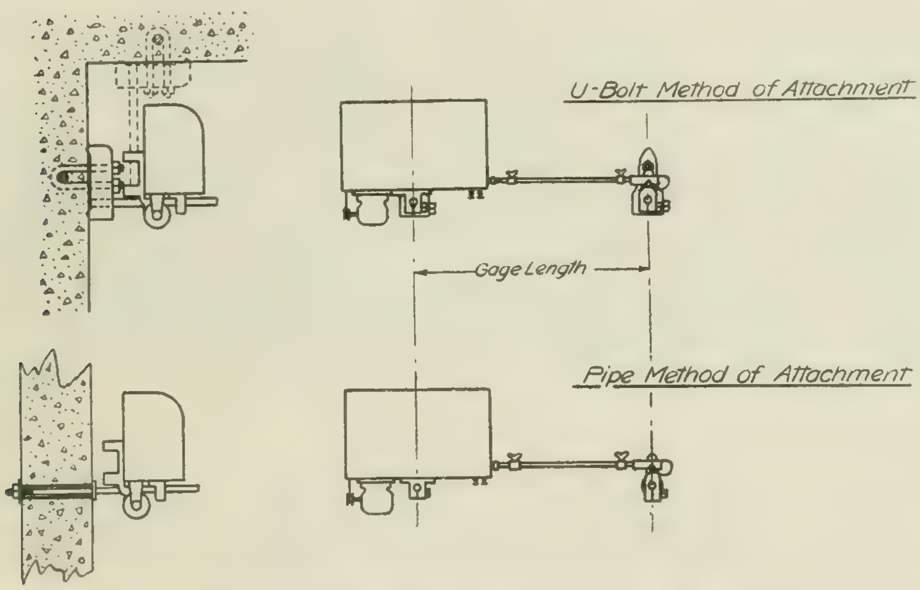


Fig. 3.—Method of Attaching the Strainagraph Used in the Steamship "Atlantus" Launching

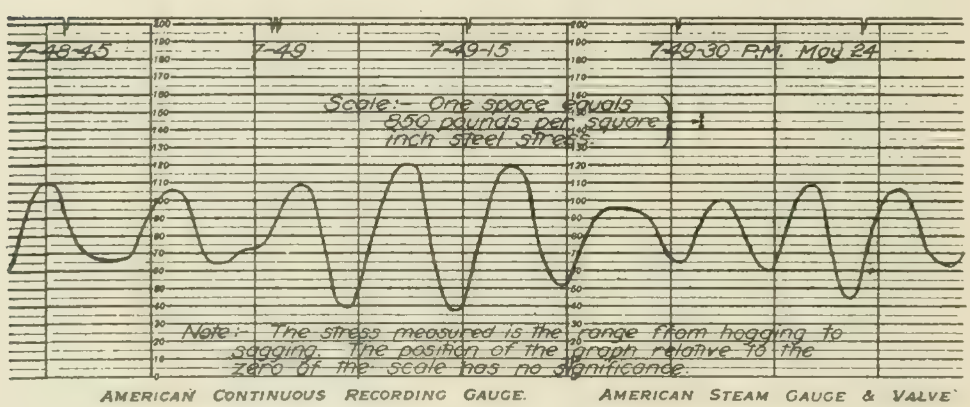
Operation of a Series of Instruments.

The motors which drive the record charts were designed to give approximately the same speed of travel to the charts of all the instruments at varying voltages, so that the speed can be varied alike in all the instruments from a central point. They are shunt-wound motors operating at all voltages between 10 and 30 and give a speed to the charts varying from 5 to 13 in. per min. They are connected in parallel to circuits from a storage battery, the voltage being regulated by cutting in the desired number of cells.

In laying out the circuits on a ship, a system of trunk lines and branches is used, with rather heavy conductors, to avoid a large drop in voltage in the long lines. Care is also taken to have the circuits approximately balanced so that all instruments operate under the same voltage.

The magnets which operate the upper pens of the series of instruments are connected in parallel to a telegraph relay instrument and a battery circuit. The primary circuit of the relay is connected through a circuit-closing clock, which gives two impulses at each minute and single impulses at each 10-second intervals. These impulses relayed to the instruments give offsets in the continuous line drawn by the magnet pens, from which the record of all the instruments can be compared for any instant of the test. A telegraph key is also connected in the primary circuit, by means of which extra impulses can be introduced to further identify the records or to record special events.

In the launching test of the "Atlantus" 18 strainagraphs and 14 pres-



Specimen Strainagraph Record

Taken on the "Faith" May 24, 1918.

Instrument located midships on a longitudinal deck beam about midway between Hatches 2 and 3 and on a line along the port side of these hatchways. The pen when travelling upward records compression; when travelling downward records extension.

Fig. 4.—Specimen Strainagraph Record.

suregraphs were thus operated simultaneously, all controlled from a convenient central point. In the tests on the 8800-ton steel ship "Westboro," in a recent transatlantic voyage, 23 strainagraphs and 12 pressuregraphs were operating at one time. In these tests over six hundred records were obtained, and throughout the operation was most satisfactory.

Strainagraph Records and Their Interpretation.

A sample record from the strainagraph is shown in Fig. 4. This is a portion of a record from the "Faith" taken during a storm. The line drawn by the magnet pen near the top of the chart will be recognized. It will be noticed that 15 seconds intervals are recorded here. In this test the identifying marks were made by closing the circuit with a telegraph key, as the circuit closing clock was not completed.

The strain record is the wavy line through the central portion of the chart. As noted under the chart an upward travel of the pen indicates a

compression or shortening of the gage length, and a downward travel indicates a tension or a lengthening.

It should be noted that the scale to which the chart is ruled is unimportant. That this is true will be apparent when it is explained that each instrument must be carefully calibrated, and unless each is provided with a chart of special ruling, a calibration factor or scale unit must be applied in making all reductions of data. The scale unit, therefore, takes care of the scale paper and a chart of any convenient ruling will serve.

On the record shown in the figure, the unit as determined by a calibration is noted. This is given for a 40-gage line and is expressed in terms of pounds per square inch steel stress for each 10 units shown on the printed chart. To obtain the change in stress between any two points, as for example for any one second or between any high point and the next low point of the curve, it is only necessary to read the paper scale for the two points and to multiply their difference by the scale unit. For example, the large change from top to bottom at about the quarter-minute point is reduced as follows:

Scale readings 120 and 37: difference $= 83$.
Stress $83 \times 85 = 7055$ lb. per sq. in.

The absence of a zero line for these measurements will have been noted from the foregoing explanation. When an instrument is attached to a ship while in motion, as is the case when shifting instruments from place to place at sea, the pen is constantly in motion and is brought to a central position on the chart before a set of records is started. For a test like the launching, the pen is brought to a convenient position on the chart and the charts run a few minutes before the ship is released. The horizontal line drawn by the pen before the ship is in motion becomes the zero line and its position on the chart can be read from the printed scale. Were it not for errors introduced by temperature changes and the possibility of the instrument being disturbed, it would be possible to attach an instrument to the ship when in quiet water and by a continuous record determine the change from still water to the extreme of a storm. The difficulties mentioned, however, together with the necessity of changing record charts, makes this impractical.

Another point in connection with these records should be mentioned here. The recorded movements are deformations and not stresses, but it is found more convenient to express the deformation in terms of equivalent steel stress than in so many millionths of an inch. Thus a deformation in the steel of the magnitude shown by the record would represent a steel stress of a certain amount. For this scale a modulus of elasticity of the steel of 30,000,000 is assumed. For any other value of the modulus, the correction can be made easily.

Calibration of Strainagraphs.

Calibration of the strainagraphs consists in determining the ratio of multiplication of the lever system. With this ratio and with the scale of

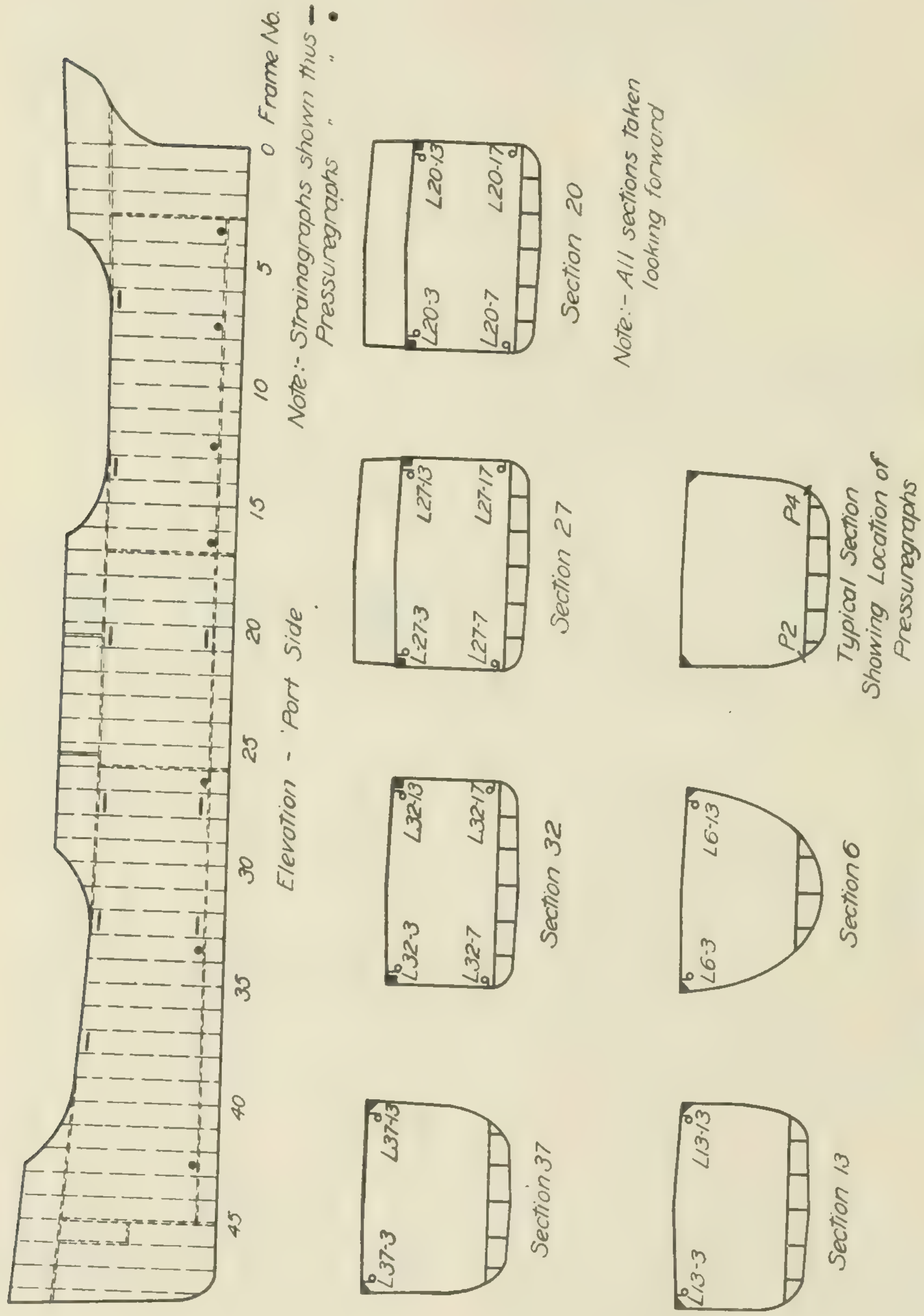


FIG. 5.—LOCATION OF INSTRUMENTS ON THE "ATLANTUS."

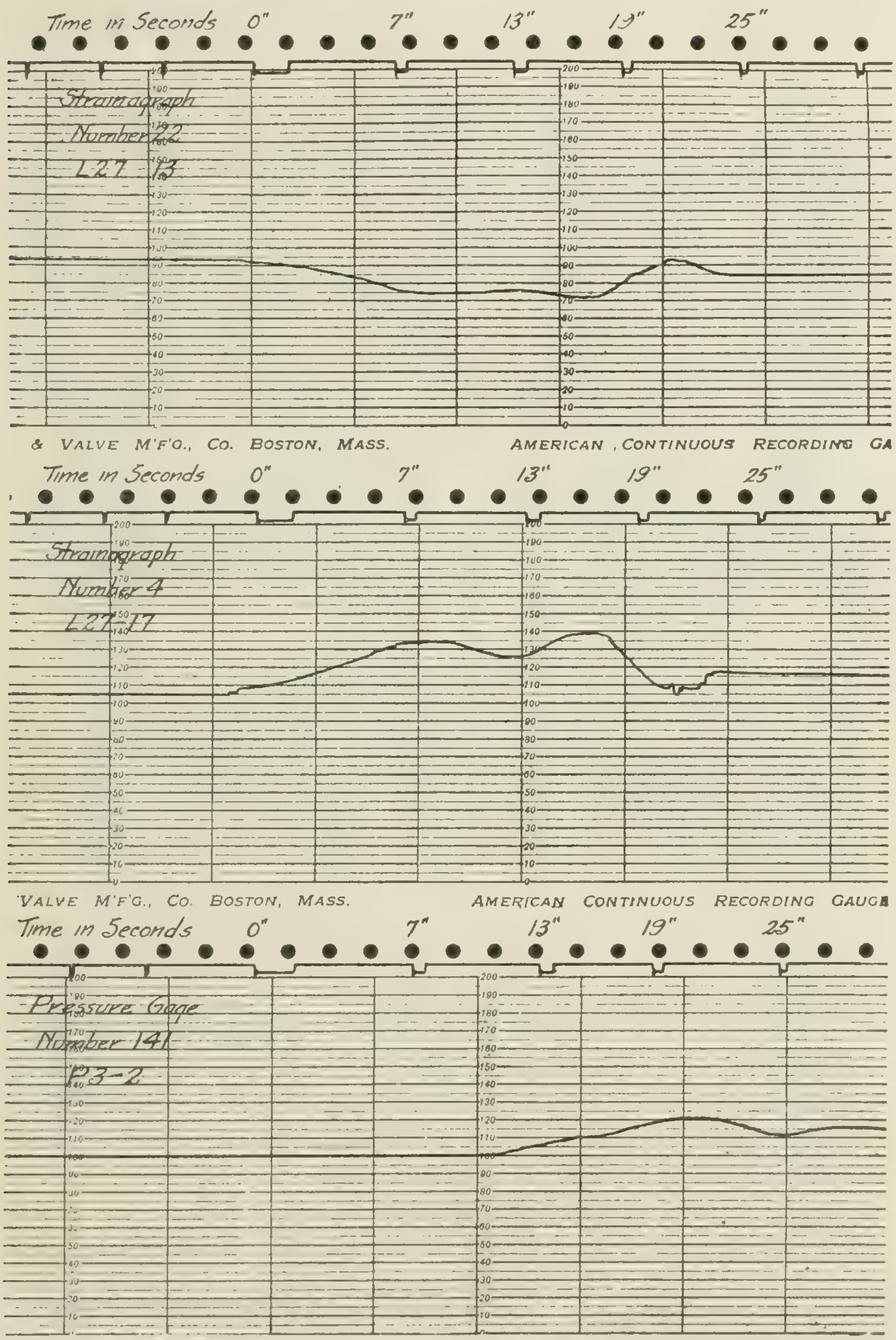


FIG. 6.—SPECIMEN RECORDS FROM THE STRAINAGRAPH.

the rulings on the paper known, what is called the Instrument Constant can be determined. The Instrument Constant is a figure which, when multiplied by the modulus of elasticity and divided by the length of the gage line, gives the scale unit, or stress in pounds per square inch corre-

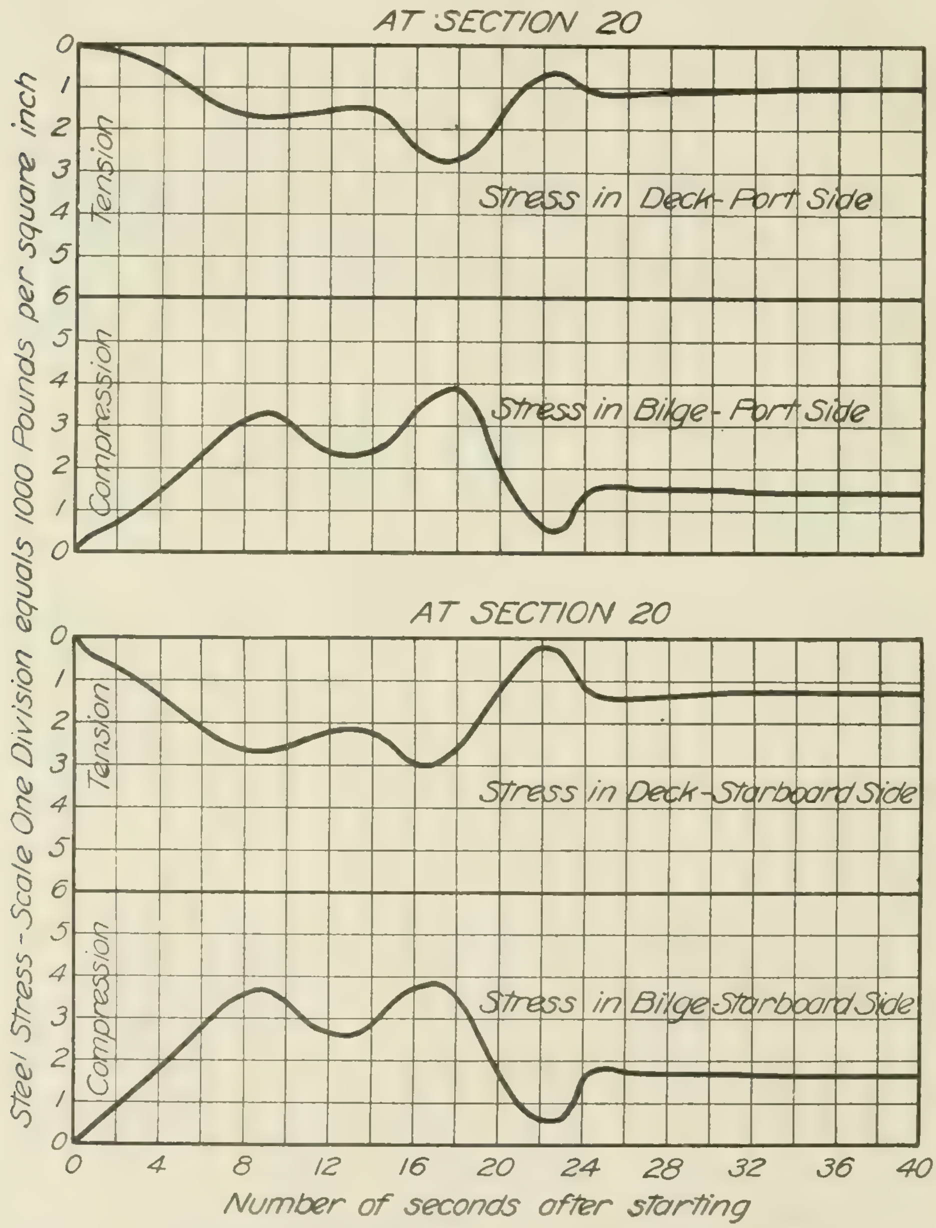


FIG. 7.—LONGITUDINAL STRESSES ON "ATLANTUS" LAUNCHING.

sponding to one unit on the paper scale. It is obtained by dividing the length in inches of one unit on the paper scale by the ratio of multiplication. For the 25 instruments used in these tests the constant varies from 0.000110 to 0.000116.

The ratio of multiplication is determined with the instrument set up

as when in service, but with the small casting in which the distance bar is pivoted clamped very lightly. An Ames gage is attached to the adjustable collar at the free end of the distance bar, so that the gage plunger bears against the strainagraph case. By slipping the lightly clamped

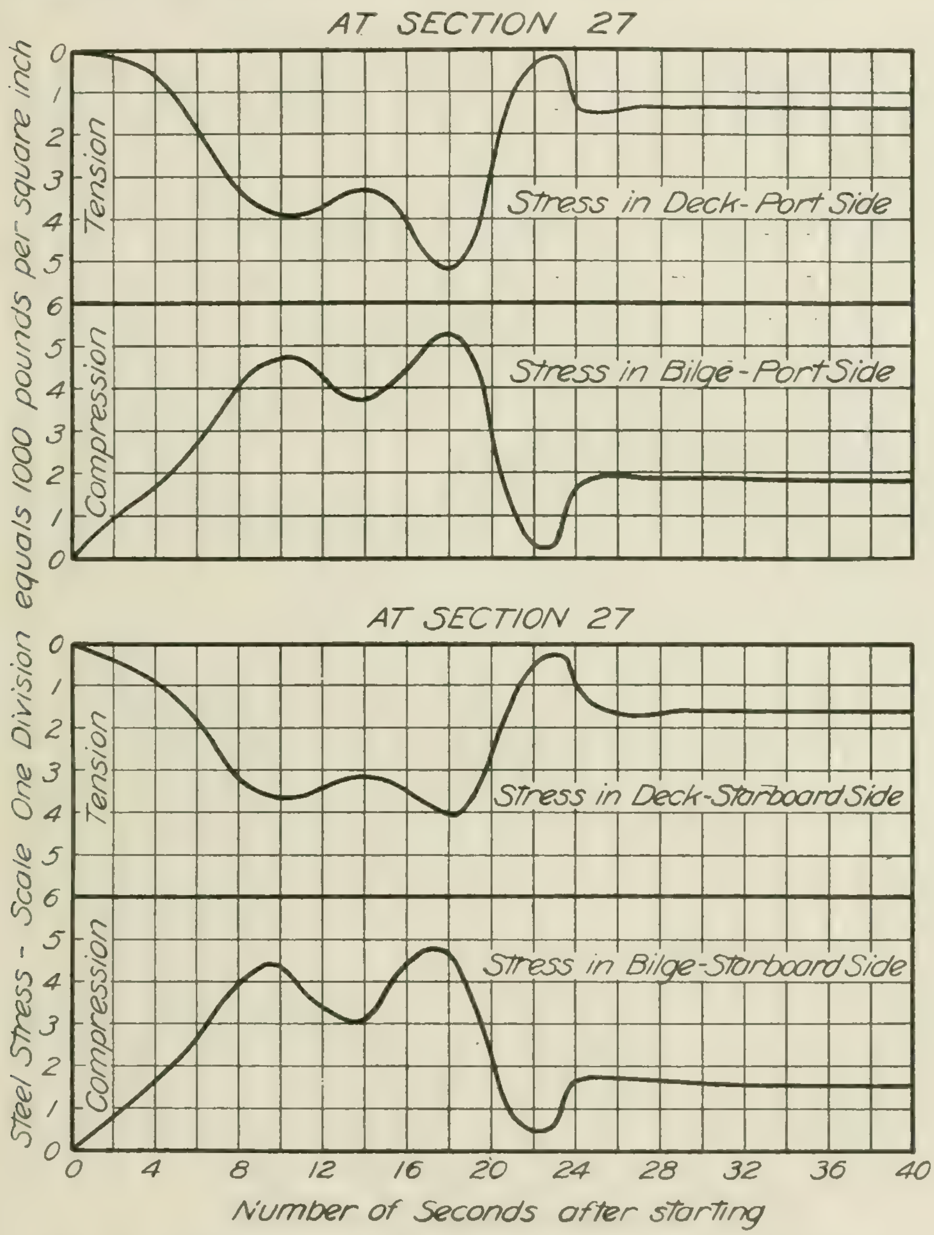


FIG. 8.—LONGITUDINAL STRESSES AT "ATLANTUS" LAUNCHING.

casting through a slight angle the pen is moved a small distance. This distance is read and the movement of the Ames gage is also noted. This gives the ratio between the movement of the pen and the true change of length in the gage line. This method of calibration has been found to be not only very convenient, but reliable and accurate as well.

Application of Strainagraph to Study of Launching Stresses.

The instrument and the interpretation of the records having been described, it remains now to show some of the results obtained from its use.

In the launching at Brunswick, Ga., of the "Atlantus," the first reinforced-concrete ship built under supervision of the Concrete Ship Section of the Emergency Fleet Corporation, the arrangement of instruments shown on the drawings of Fig. 5 was used. As this was an endwise launching, in which it was desired to obtain longitudinal stresses, the significance of the arrangement will be apparent.

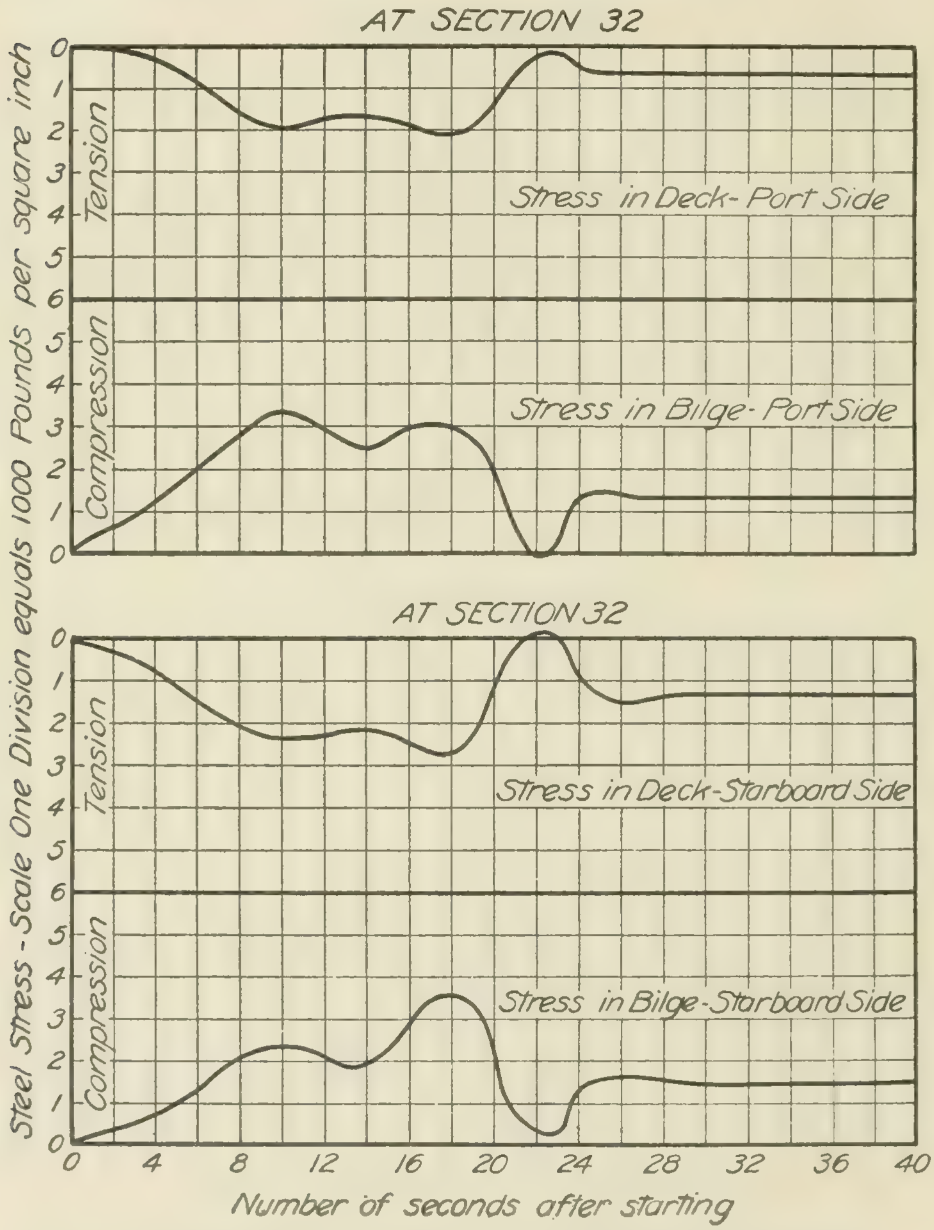


FIG. 9.—LONGITUDINAL STRESSES AT "ATLANTUS" LAUNCHING.

Sample records for the strainagraphs at two points during this launching are shown in Fig. 6. It will be noted from the location number given on the records and from the key diagrams of Fig. 5 that these two instruments were located between frames 27 and 28, on the starboard side. The upper record is from the instrument at the deck and the second record from the instrument on the shell just above the inner bottom. The lower record shown in Fig. 6 is one of the pressuregraph records taken during this test.

From the descriptions given previously the meanings of these records will be clear. The straight portion of the line made by the marking pen, at the left of the figure, is the record made before the ship began to move. The stress at any instant subsequent to this is referred to this line as a zero. The record for the deck location will be seen to be tension throughout, and that for the lower instrument, compression.

On Figs. 7, 8, and 9 are plotted the results from the strainagraphs at the three sections where instruments were located at both deck and bottom. In preparing these curves the deformations have been reduced to equivalent steel stresses and have been corrected for the positions of the instruments to show the stresses at the extreme top and bottom. In making this correction straight-line variation of stress has been assumed.

It is not the purpose of this paper to present a full discussion of the results of this launching, for the combined strain and pressure records provide data for a study too extensive to include here. Therefore, only the briefest discussion will be attempted. The pressure records will not be given, and only one set of results derived from them will be shown for the purpose of illustrating their use. These results are given in Fig. 10 and show the draft for four successive seconds just before the bow passed off the ways. On this figure the drafts are plotted as ordinates downward from the base line. The full line shows the actual water surface as obtained from the records, and the dotted line shows the plane of the still-water surface. On the outline of the ship at the top the actual water surface for the 19th second is drawn to the same vertical scale as the ship. It will be noted that the slope of the dotted line referred to the base line, and, expressed in the proper scale, gives the direction of travel of the ship relative to the water surface. From these and similar curves for other periods the buoyancy has been determined for any instant. The moments given in Fig. 14 were calculated from the buoyancies determined from these curves.

An examination of Figs. 7, 8, and 9 shows that hogging stresses (that is, tension in the deck) began to develop as soon as the hull started to move and these increased until about the 9th second. Following this will be seen a partial return to a condition of no stress and then a further increase to about the 18th second. From the 18th to the 22d second there was a sharp decrease in the stresses to very nearly zero at all points and a quick return to the condition in which it remained in quiet water.

The stress during the first 12 or 15 seconds was due to a slight vertex

in the ways, due to irregularities in construction, over which the hull had to pass. From the 16th second the stress was due to the hogging of the ship over the end of the ways. This reduced to almost no stress from the 18th to the 22d second as the buoyancy at the stern increased. From the 25th second to the end of the record the hogging stresses are due to the normal hogging moment in the hull.

The variation in stress along the ship at the 18th second is shown in Fig. 11. The data for these curves were obtained from the curves of Figs. 7, 8, and 9, and similar curves for other points. This figure shows also the position of the ship on the ways and the observed drafts for that instant. Similar curves for the 22d and 32d seconds are shown in Figs. 12 and 13. These show respectively the conditions at the time of lowest stress and under the final hogging moments.

The most interesting feature of this test is the comparison of the resisting moment as determined from the strainagraph records with the applied moments as calculated from the observed drafts. This comparison is shown in Fig. 14 for the three sections where measurements were made at both top and bottom. The calculated moments were obtained by the usual calculations based on known weights and buoyancies.

In calculating the resisting moments from the strainagraph data the same assumptions were made as were used in the preparation of the design of the ship. These include those usual in the common theory of flexure and the further assumption that the bulwarks and bridge deck did not add to the resisting moment. It should be noted here that the bridge deck and bulwarks were cast separately from the hull and expansion joints were provided about 25 ft. apart. All the indications from the data point to the reasonableness of these assumptions and until further data is obtained from the behavior of this ship at sea it seems fair to accept them.

These calculations for resisting moment were made for a beam of homogeneous section, giving to the steel area a weight equal to the ratio of its modulus of elasticity to that of the concrete. Tests of 31 cylinder specimens from various parts of the ship show an average value of 2,560,000 for the modulus of the concrete. A ratio of 12, therefore, was used on these calculations both for weighting the steel area and in reducing deformations in the concrete to equivalent stresses. The assumption as to the homogeneity of the section is considered fair, as the maximum tension did not exceed 500 lb. per sq. in., and there was no evidence of tension cracks after the launching.

Referring now to Fig. 14: The real significance of the agreement between the two sets of curves will be appreciated. These curves represent the moments in the ship during the critical period of launching as determined by two independent sets of measurements of a widely different character. One set of curves gives the calculated applied moments based on water pressure observations, and the other set of curves gives the measured internal moment of resistance based on measured deformations in the hull.

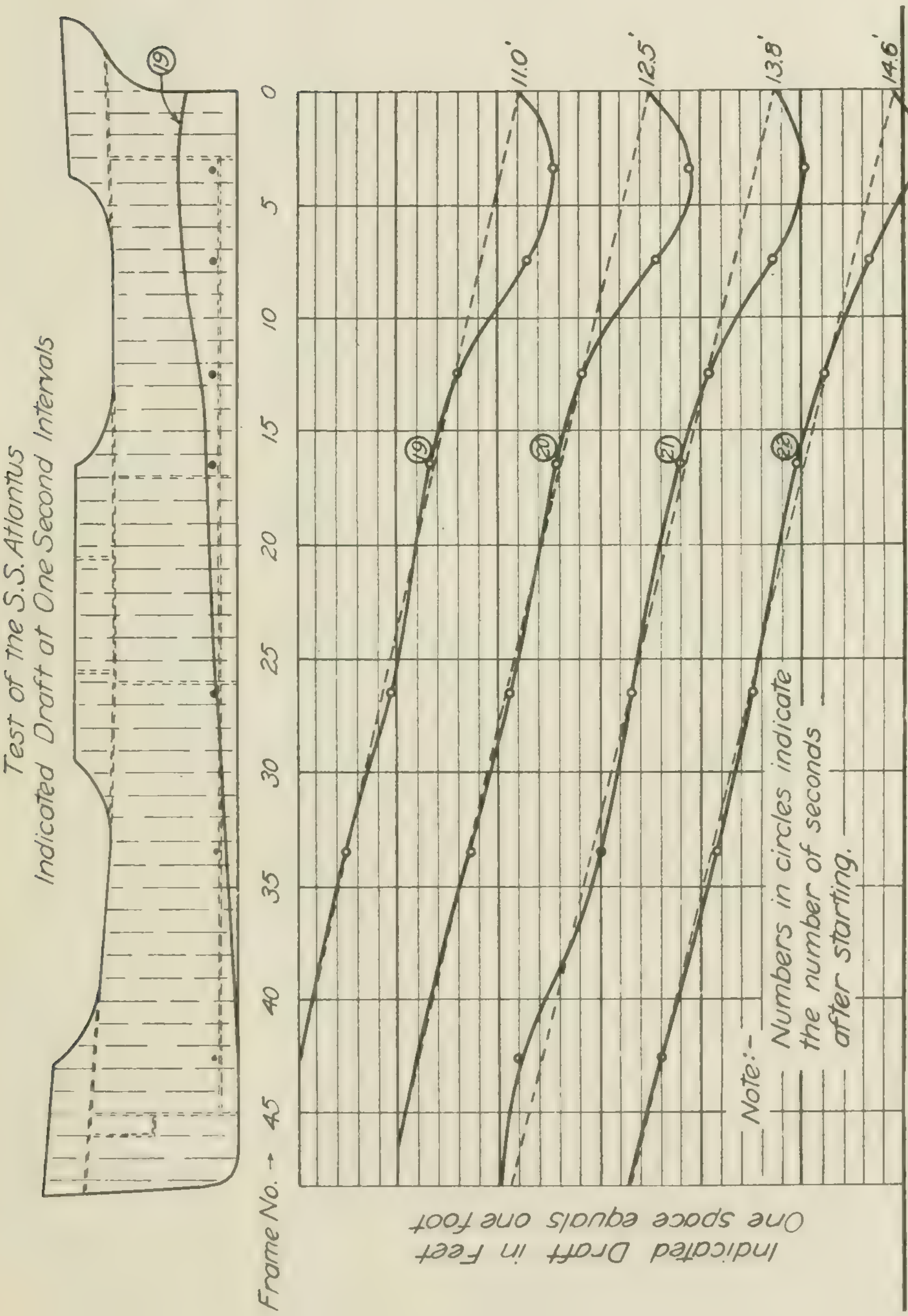
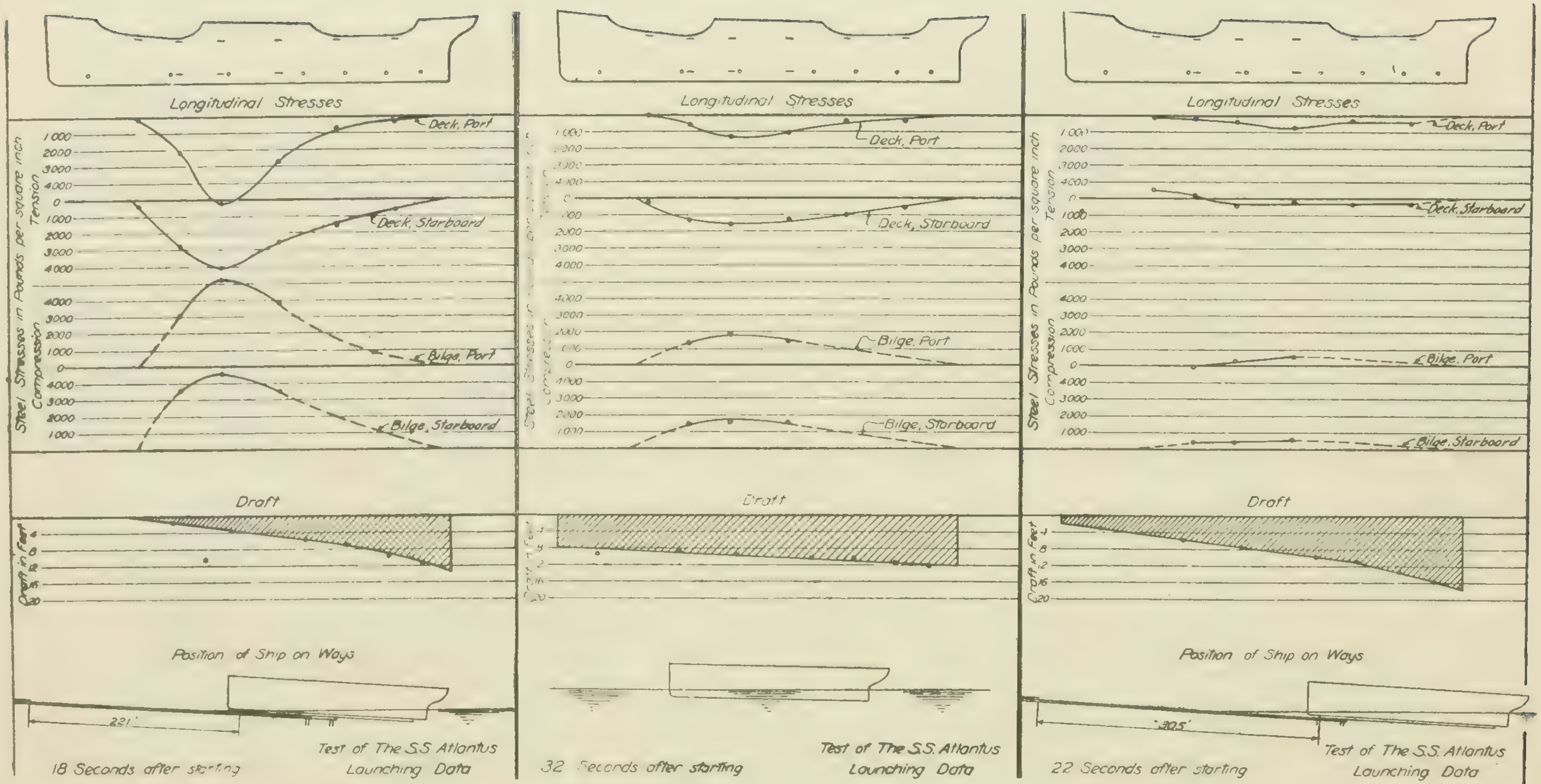


FIG. 10.—INDICATED DRAFT AT ONE-SECOND INTERVALS OF STEAMSHIP "ATLANTUS" DURING LAUNCHING.



FIGS. 11-12-13.—LAUNCHING DATA AT DIFFERENT PERIODS OF THE LAUNCHING OF THE "ATLANTUS."

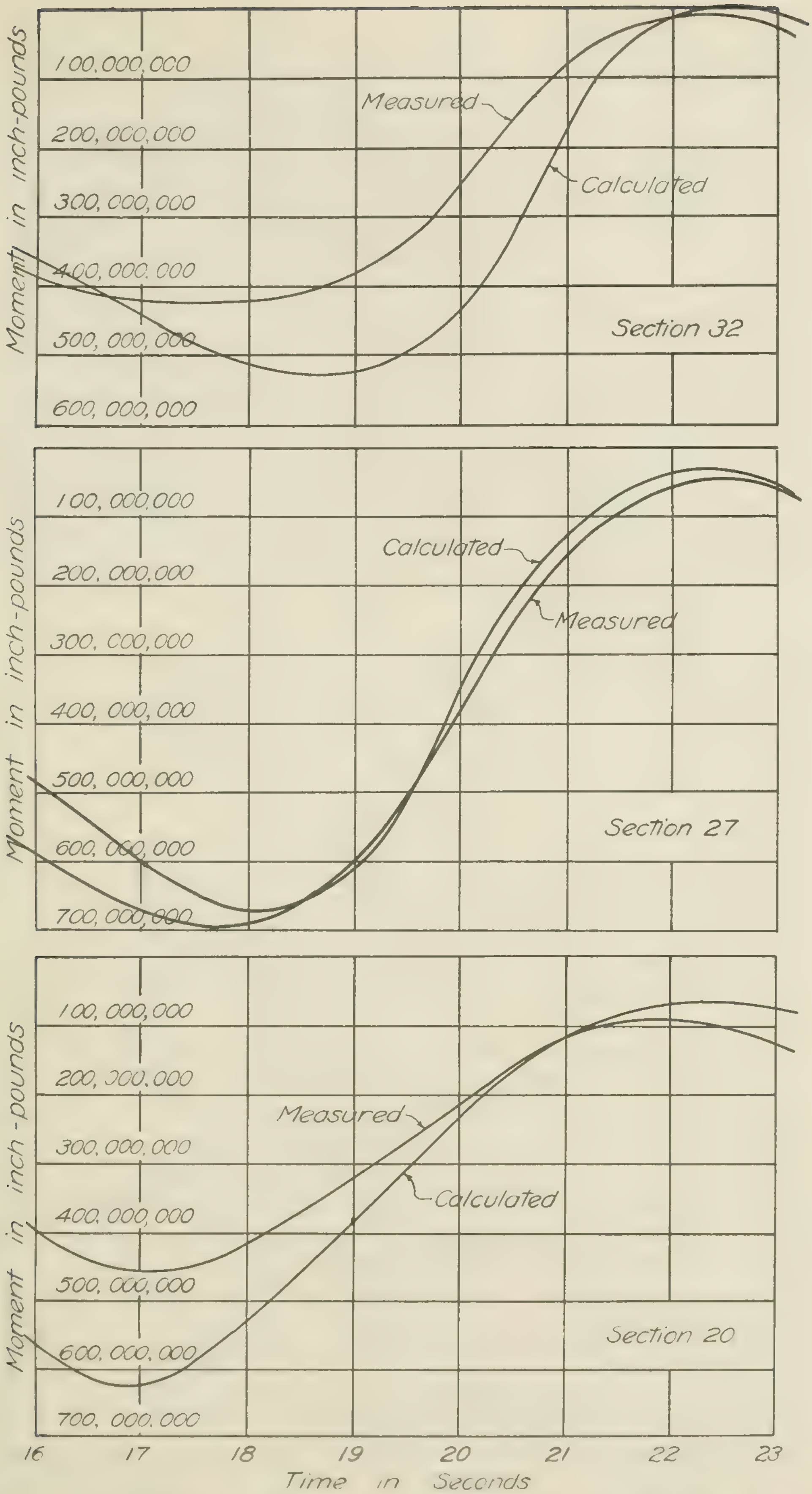


FIG. 14.—COMPARISON OF MEASURED AND CALCULATED STRESSES ON THE "ATLANTUS" DURING LAUNCHING.

It will be noted that in form the curves for each of the three sections agree remarkably well, also that the agreement in values for section 27 leaves nothing to be desired. In regard to section 20, it can be stated that the calculated moments for the 16th, 17th, and possibly the 18th seconds, are somewhat too high, for the arrangement of pressure instruments was such that they did not record the full buoyancy at these periods. This would indicate better agreement than appears on the curve. In regard to section 32, it may be said that the curve of applied moments at the forward end of the hull cannot be determined as accurately as elsewhere because of the uncertainty in the distribution of the reaction from the ways.

In the light of these considerations it may be stated that the results are very satisfactory and show a high degree of reliability in the instruments. They also give a fair indication of what may be expected in future tests where it will be possible to eliminate some of the uncertainties encountered here.

PLASTICITY AND TEMPERATURE DEFORMATIONS IN CONCRETE STRUCTURES.

By S. C. Hollister.*

In the fall of 1917 a series of tests on flat-slab structures was carried on by Prof. W. K. Hatt at Purdue University at the instance of the Corrugated Bar Co. of Buffalo. At the outset a study was made by the writer of means of eliminating temperature and plastic effects from the observed strain-gage readings. The present paper is a report of this study.

The paper deals first with a series of temperature observations made upon the unloaded test structure, and secondly, with a series of observations of plasticity upon a coupon slab and upon one of the test structures under load.

Study of Temperature Deformations.

Review of Present Methods of Making Temperature Corrections.—There are several methods employed at the present time for making corrections due to temperature change in the data obtained from strain-gage measurements upon reinforced-concrete structures under load. These methods may be described briefly as follows:

(1) The temperature of the atmosphere at various times during the taking of observations with the strain gage may be noted. A coefficient of expansion for the steel or for the concrete, either previously determined or assumed, is then employed to compute a thermal deformation corresponding to the change in temperature of the surrounding air. These computed thermal deformations are then applied to the original observations made with the strain gage.

(2) A reinforcing bar similar to the reinforcement in the structure may be embedded in a block of concrete to a depth corresponding to the average depth of the reinforcement upon which strain-gage measurements are being taken. If the structure is of the flat-slab type, the block of concrete is usually of the same thickness as the slab, and cast at the same time. During the observations on the structure occasional readings are taken upon this so-called calibration bar, and the variations of deformation noted on the calibration bar represent the correction due to temperature change, which is then applied to the original strain-gage readings.

There have been conditions where either one of the above methods of correction of the strain-gage readings has proved satisfactory, but it is certain that neither method can be applied generally for the correction of strain-gage observations. The first method is in general objectionable because: (1) the structure does not always assume the temperature of the air surrounding it; (2) there is usually a considerable lag in the rate

* Research Engineer, Corrugated Bar Company, at time of tests.

at which the structure assumes the temperature of the air; (3) any thermal coefficient predetermined, or any simple member will not give any indication of the flexural stresses actually set up in the structure due to the change in temperature of its various parts; and (4) because it is not at all settled that the "thermal coefficient" for any given gage length on an actual structure remains a constant through all stages of elastic deformation of the loaded structure.

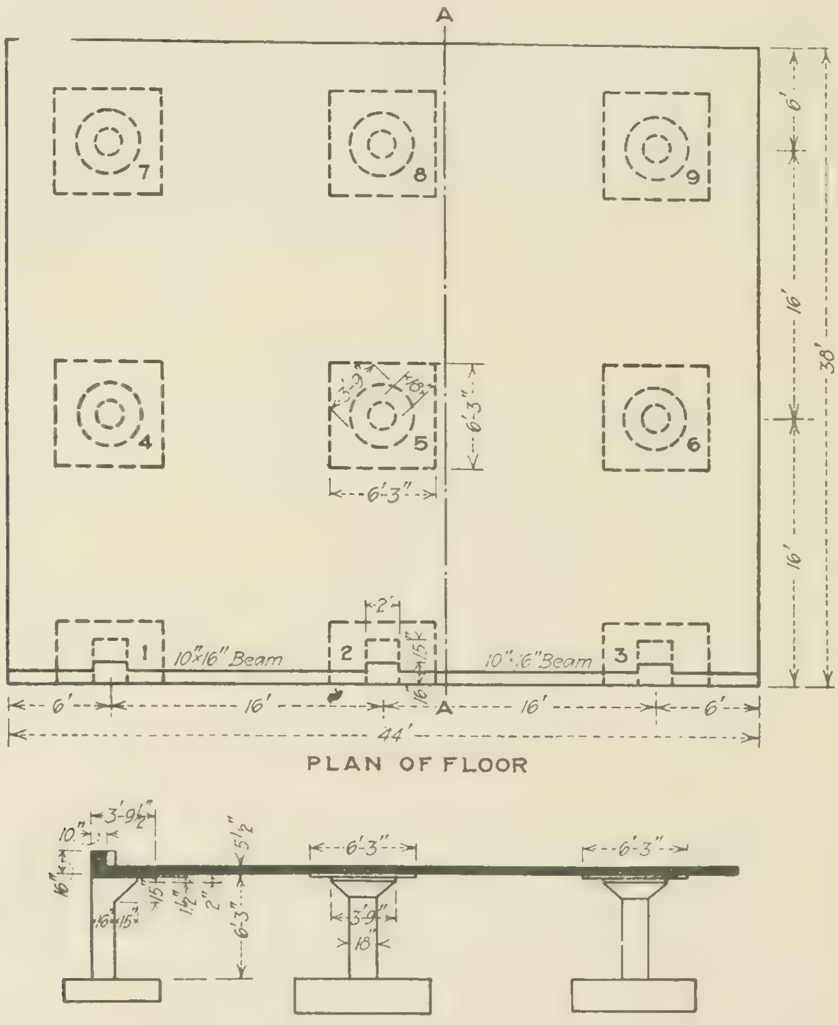


FIG. 1.—FLAT SLAB STRUCTURE TESTED.

The second method presents the difficulty that the calibration bar does not always assume the same temperature as the structure, nor does it necessarily assume it at the same rate. It has also the same objection as that mentioned for the first method, in that it is a simple member in which linear expansion may occur freely, and therefore it gives no indication of the secondary stresses which actually arise in the structure, due to changes in temperature in its various parts.

Before beginning the tests of the behavior of the flat-slab structure shown in Fig. 1, under various increments of loading, a study of the behavior of the structure was made under variable temperature conditions. Gage lines upon the steel and upon the concrete were selected at points of critical stress, with a view to determining if possible a law of behavior of the structure under temperature variations for the purpose of reducing all of the strain-gage data during the load tests to a given temperature datum.

Description of Structure Under Test.—Fig. 1 shows the general dimensions of a typical test structure. It is a Corr-plate slab designed by the Corrugated Bar Co. for a live load of 150 lb. per sq. ft. The flat slab floor is 38 x 44 ft. in plan and is supported on columns spaced 16 ft. on centers either way. On three sides of the structure a 6-ft. overhang of the slab is provided to give a negative moment over the side margins of the square panels of the floor, and on the fourth side a 10 x 16-in. lintel beam imitates the condition which obtains in wall panels of a commercial building. The slab is 5½ in. thick with 2 in. additional at the drop panels.

Description of Observation Taken.—Gage lines upon which maximum stresses were expected to occur during the loading test were selected on the upper and lower faces of the slab (Fig. 2). On the upper faces of the slabs these readings included gages across the edge of the column capitals and across the margin of the panel. On the under face readings were taken at the center of the panel in both directions; on the line between columns and half way between them, and on gage lines on the concrete on the under face of the drop adjacent to the edge of the column capital. Readings were also taken on the inside face of one of the wall columns immediately below the bracket. Readings were taken on all of these gage lines at periods of high and low temperature over the space of about three days. Care was taken to read at a time at which the temperature had remained constant for an hour or more, in order to prevent the introduction of the thermal lag in the slab.

Readings were taken on a "calibration bar" and a "standard bar" during the observation of the gage lines mentioned above. The "calibration bar" consisted of a bar of reinforcement similar to that used in the slab and was embedded in a block of concrete having a thickness similar to that of the test panel, a distance from the surface of the concrete equal to that in which the bars were embedded in the flat slab. The block of concrete containing the bar was supported above the ground immediately underneath the test structure, where it was protected from the direct exposure of the sun, yet was accessible to all air currents passing under the structure. The "standard bar" consisted of a bar of Invar nickel steel. The coefficient of expansion of this standard bar was previously determined to be 0.0000009. Readings upon it were taken for the purpose of ascertaining to what extent strain gages themselves may have expanded or contracted during any series of observations.

In conjunction with the above readings, the following temperatures were observed at various points in and about the slab:

(1) *"Exposed Air Temperature."*

This was observed by a thermometer hanging about 5 ft. above the slab and exposed to the sun and to air currents.

(2) *"Shaded Air Temperature."*

This temperature was observed on a thermometer hanging underneath the slab, in the shade, but exposed to air currents.

(3) *"Calibration Bar Temperature."*

A thermometer was embedded in the concrete block in which the calibration bar was embedded. The bulb was at the same depth as the calibration bar and adjacent to it.

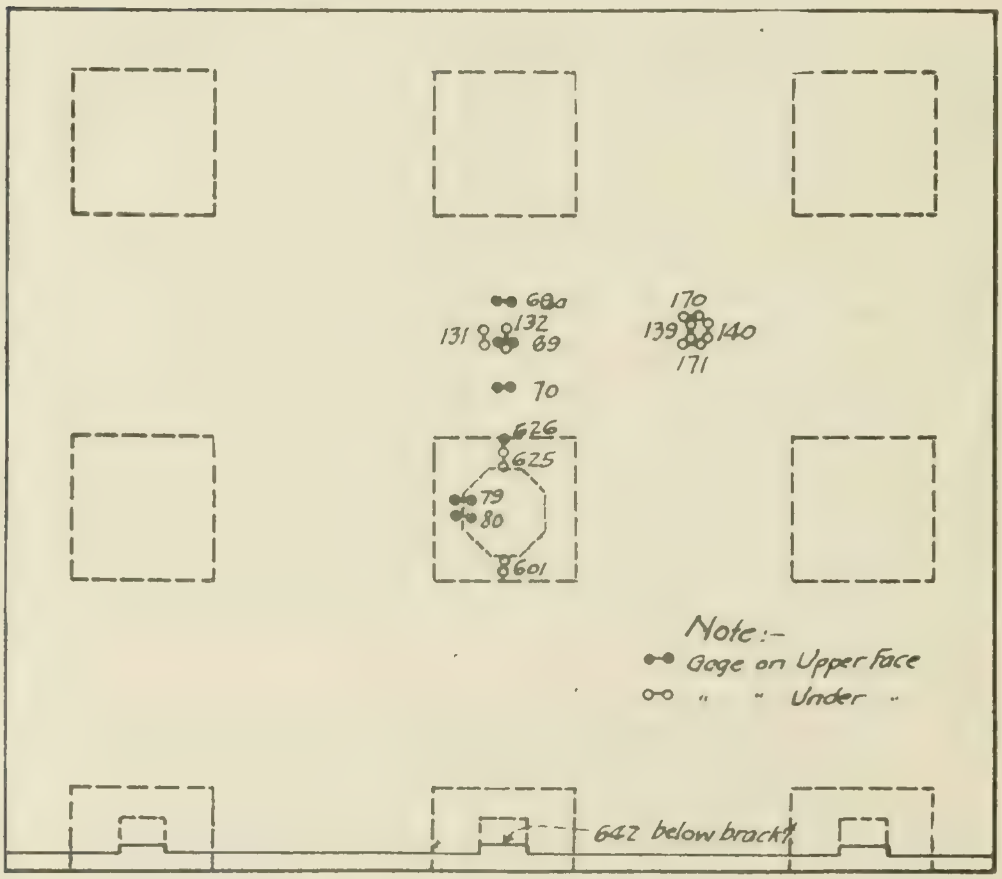


FIG. 2. GAGE POINTS ON TEST FLOOR.

(4) *"Slab Temperature, Top."*

A thermometer was set with its bulb barely embedded in the upper face of the slab.

(5) *"Slab Temperature, Bottom."*

A thermometer was set in a hole drilled from the top of the slab to within ⅜ in. of its bottom face.

Temperature observations made at the various points noted above were plotted in Fig. 3. The range of exposed air temperature was approxi-

mately 40° over the period of four days in which these readings were taken. Care was taken to select a period for reading which was as far as possible one of entire descending and ascending temperature. The lines connecting the points of observation do not represent the actual temperature variations between observations, but merely connect each set of readings in sequence.

It should be noted from this set of readings that the temperature of the air above and below the slab was very nearly the same, and that the difference in temperature of the slab and the temperature of the surround-

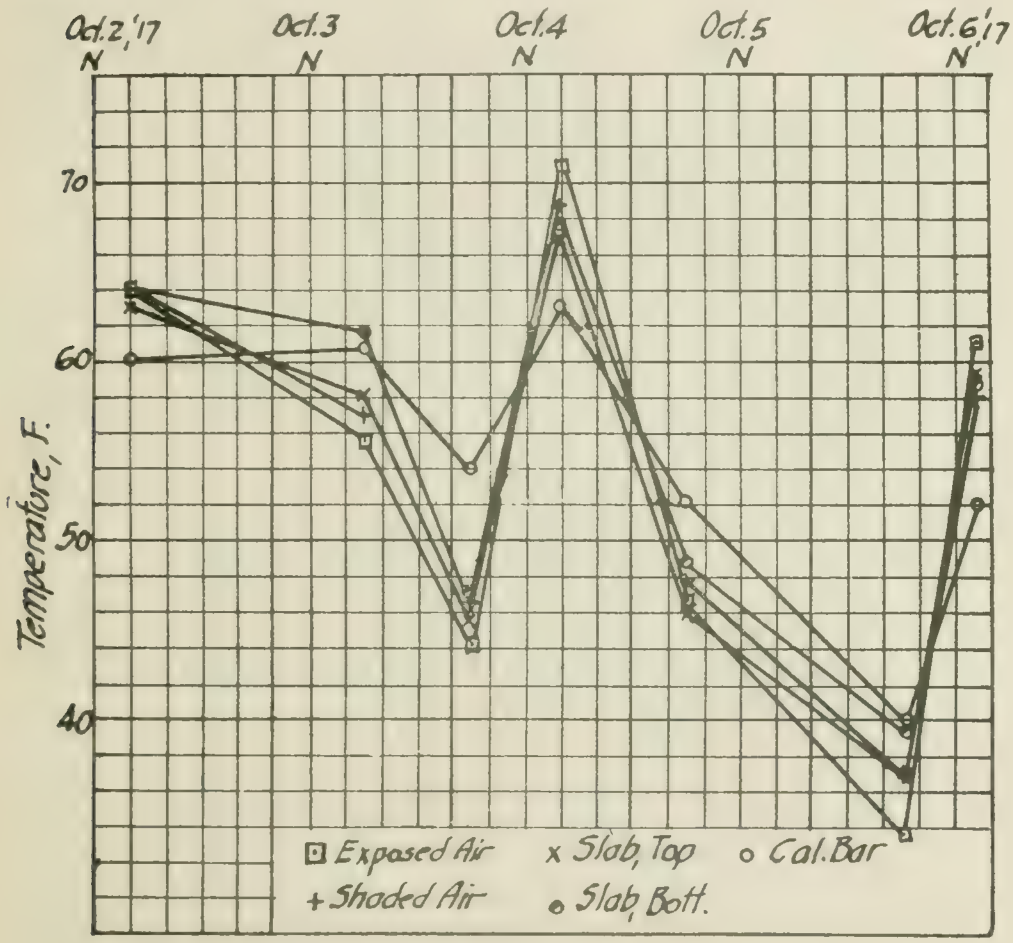


FIG. 3.—TEMPERATURE OBSERVATIONS MADE AT VARIOUS POINTS.

ing air was relatively small. It is important to notice also that the thermal susceptibility of the concrete block in which the calibration bar was embedded was not nearly as great as in the case of the flat slab structure itself. Assuming the same coefficient expansion of the steel embedded in the flat slab structure and for the calibration bar, the conclusion would at once be that the thermal expansion of the reinforcement of the test structure was not duplicated in the thermal expansion of the calibration bar, and hence the calibration bar would not offer adequate correction for

the temperature effect included in the strain gage observations made on the test structure itself.

Fig. 4 shows the actual strain-gage readings, plotted horizontally against time and vertically against deformations. These readings are grouped according to the location of the gage lines on which they were taken on the test panel. It should be remembered that these readings were taken when the slab was entirely unloaded, so that any deformation which occurred was due solely to temperature conditions. Superimposed

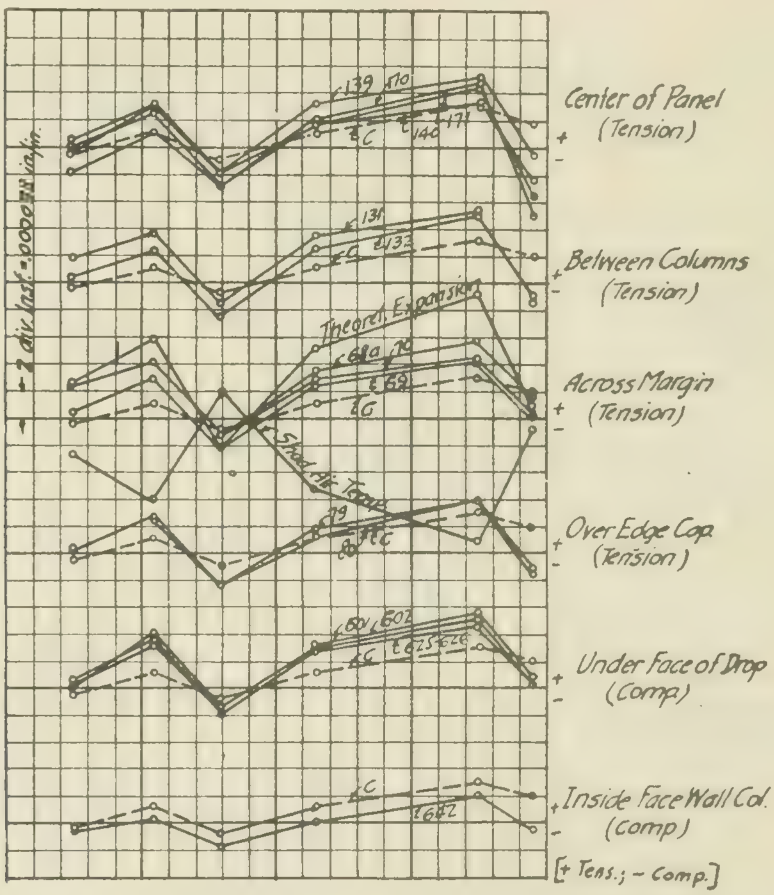


FIG. 4.—STRAIN-GAGE READINGS—TIME VS. DEFORMATION.

upon each of these groups will be noted the curve labeled "C," which is the result of readings taken on the calibration bar. It will be noted that the calibration curve "C," while of the same form as the other curves, is of a decidedly different magnitude, which indicates a difference in thermal susceptibility between this particular calibration bar and the test panel.

On the group of readings taken across the panel margin will be noted a curve of shaded air temperature. The curve in general is similar in form to all of the strain gage readings shown. This variation in temperature, when transformed into thermal expansion in a naked steel bar whose length

equals that of the gage of the instrument, is shown by the curve marked "Theoretical expansion." The coefficient of expansion used in arriving at this curve was 0.0000065..

A considerable difference may be noted in the magnitude of the thermal deformations on the gage lines and the theoretical thermal expansion computed as described above. This would indicate that, based on the same temperature, the coefficient of expansion on the gage lines of the structure was smaller than that for steel.

It seemed clear to the writer that the calibration bar did not offer adequate temperature correction nor did the computation of theoretical expansion based on either the air temperature or the slab temperature furnish sufficient basis for elimination of temperature effect. From a study of readings taken on the gage lines of the structure, it was noted that the temperature effect for all readings on reinforcement irrespective of location of the slab was practically the same. Similarly, it was noted that the variations due to temperature on the gage lines on the steel reinforcement and gage lines on the concrete, with the exception of readings on the wall column, were practically the same. It was, therefore, decided to use these readings on the structure itself and to determine a so-called coefficient of expansion for it in terms of some one of the observed temperatures. For this purpose, the air temperature beneath the slab was chosen since it more nearly represented the temperature within the slab itself than any of the other outside temperatures, and was the easiest to obtain. In this manner the coefficient of expansion of the slab reinforcement was found to be slightly over 0.000004 per degree change in the shaded air temperature.

Fig. 5 shows representative gage lines from each group of gages located at different points on the slab. Superimposed on the first of these are the results of various methods of correction for temperature deformations. It will be noted on Gage 139 that the measurement on the calibration bar reduced the temperature effect by about half, whereas the air temperature, either above or below the slab, reduced the curve to practically a straight horizontal line. A correction based on the air temperature below the slab was applied to all other gage readings shown in this figure, indicating that, for any position on the slab and for either steel or concrete readings, the effect of temperature was reduced to practically zero.

Application of Correction Coefficient.—The method of applying the correction to the original strain-gage readings was to determine the average elongation of the 10-in. gage line per degree change in shaded air temperature. All of the readings previously shown were plotted between deformation and temperature. The average slope of these curves indicated the variation of the instrument, reading 0.2 division per degree, corresponding to a unit change in length of 0.000004 per degree change in temperature. This coefficient was applied to the temperature readings simply by multiplying the coefficient by the degrees change of temperature for the gage line at the particular time in question.

Panel Deflections.—Panel deflection readings corresponding to the

strain gage readings just referred to, were made for the first series of temperature readings. Results of these readings were erratic, due in large part to the temperature changes, and deformations caused by differences in

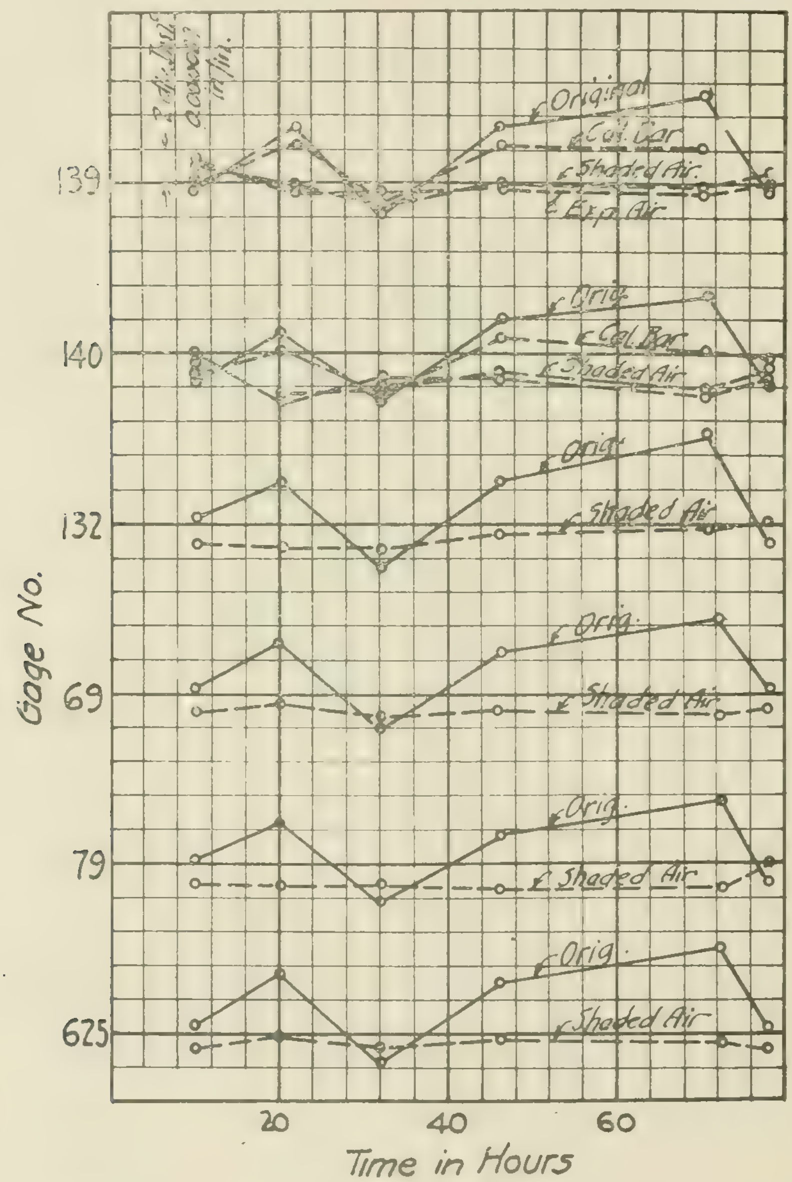


FIG. 5. REPRESENTATIVE GAGE LINES AT DIFFERENT PARTS OF SLAB.

humidity, both of which affected the wooden bent upon which the deflectometers were mounted.

OBSERVATIONS OF PLASTICITY.

Test of Coupon Slab.—A small concrete slab, 27 in. wide, 5½ in. thick and reinforced with 0.5 per cent of steel, was tested in the laboratory at

Purdue University under a third point loading and on a 75-in. span. The purpose of this test was to determine what percentage of the external moment of the slab was accounted for by a moment computed on the basis of the measured steel stresses. This information was to be applied to certain data obtained from the flat slab test structures in the interpretation and computation of moment distribution. The slab was loaded on a time schedule approximating closely the schedule of loading of one of the test structures. Since the test was conducted indoors, the effect upon the readings taken on this coupon slab due to temperature was legitimately negligible. Readings were taken before and after the application of each increment of loading to determine the extent of plastic deformation and deflection.

Plasticity Defined.—Plasticity is differentiated from elasticity in that it is a property of a material which does not exhibit a return of dimension or shape after the stress causing the change of dimension or shape has been removed. It causes "permanent set."

Plastic deflection may be defined as that deflection of a beam or other transversely loaded member which does not disappear on the removal of the load causing the deflection. For most building materials it is only a part of the total deflection of such a member.

Plastic deformation is in general that deformation which does not disappear upon removal of the stress which produces it. In most building materials it is only a part of the total deformation caused by a given stress. The term will here be used to indicate the plastic deformation observed on a gage line by means of the strain gage.

Plastic Deflection.—Fig. 6 shows the deflection of the coupon slab measured at the center of the span and plotted against the applied load. The stepped curve is the curve of the original readings and indicates at each step the accumulation of deflection due to continued application of that load. The curve is the average of readings taken near either edge of the slab.

If each consecutive step were eliminated from the original curve, the result would be a curve of deflection occurring during the application of load without regard to the interval of time existing between these applications. It will be noted that above the 8,000-lb. load this curve, as shown in the figure, is practically straight. The curvature at approximately 8,000-lb. load is due to the change in rate of deflection caused by the correcting of the slab on the tension side.

The curve marked "Summation of Plastic Deflections" is an accumulation of the various steps of the original curve, and the average curve through the points indicates in general the rate of accumulation at different loadings. It will be noted that this rate changed at the time when the cracks began to form on the tension side of the slab.

In order to indicate the importance of the elimination of plastic deflection in such a test, a curve was drawn to show the per cent of the total deflection, which was plastic deflection only. This curve would have indi-

cated a greater percentage of plastic effect had the time interval of duration of load been increased.

Plastic Deformation.—Considerable plastic deformation was observed on the gage lines on both steel and concrete on the coupon slab. The results of these observations are shown in Fig. 7. For both steel and concrete, the total deformation for the gage has been plotted as well as the plastic deformation only and the elastic deformation only. In general, the plastic deformation was of about the same magnitude as the elastic deformation.

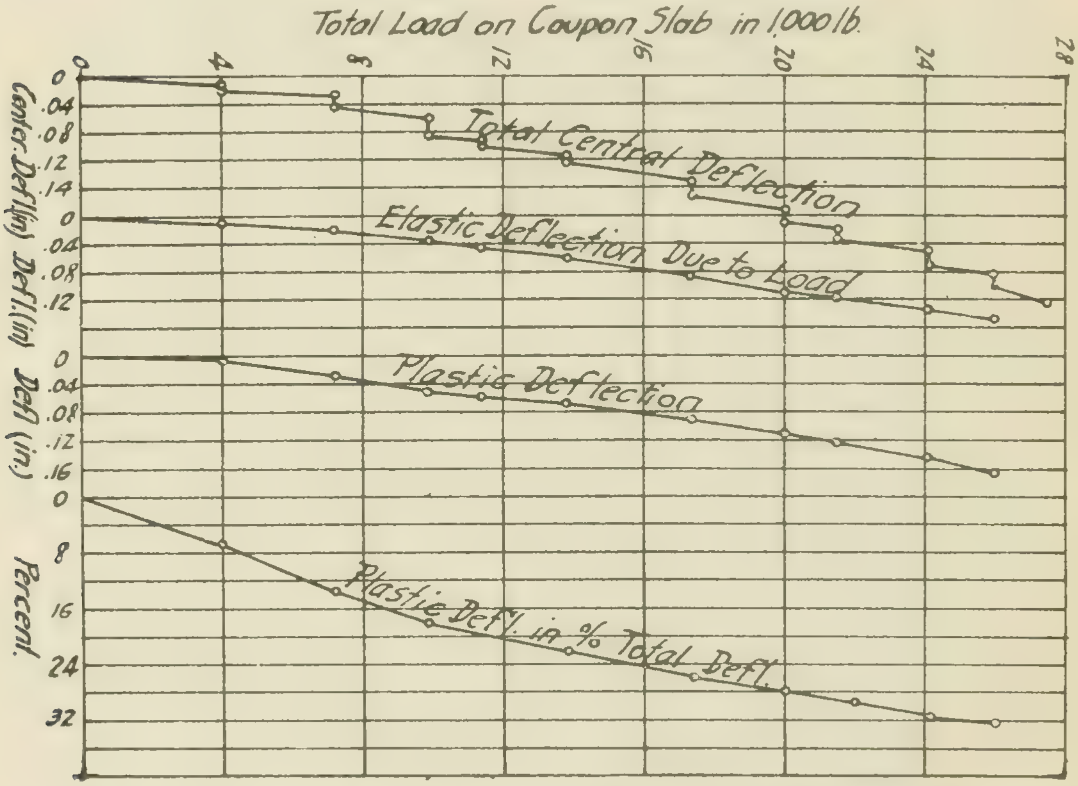


FIG. 6.—DEFLECTION OF COUPON SLAB AT CENTER MEASURED AGAINST LOAD.

The plastic defect would, of course, have been more pronounced had the interval of time of duration of each load been increased.

PLASTIC EFFECT AS OBSERVED IN THE TEST STRUCTURES UNDER LOAD.

Considerable plastic effect, both in deflection and deformation during the progress of the testing of the flat slab structure under load, was apparent. Increase in the period of duration of load gave opportunity for the steady appreciable increase in plastic effect and it was, therefore, necessary to limit as far as possible the time required to apply each increment of load as well as the time in which each increment was in place before the following increment was applied.

It was evident from the tests on these structures where a line could be drawn between elastic lag and plastic effect. In this sense, the writer considers elastic lag to indicate the delay of elastic deformation corresponding to a given load before it has reached its full magnitude. This could be readily determined by removing the load causing the particular deformation or deflection, but such a procedure was not possible in the course of the test. The practice, therefore, of reading deformations before and after the application of each increment of load was followed in each case where the increment remained in place longer than 24 hours before the following increment was begun.

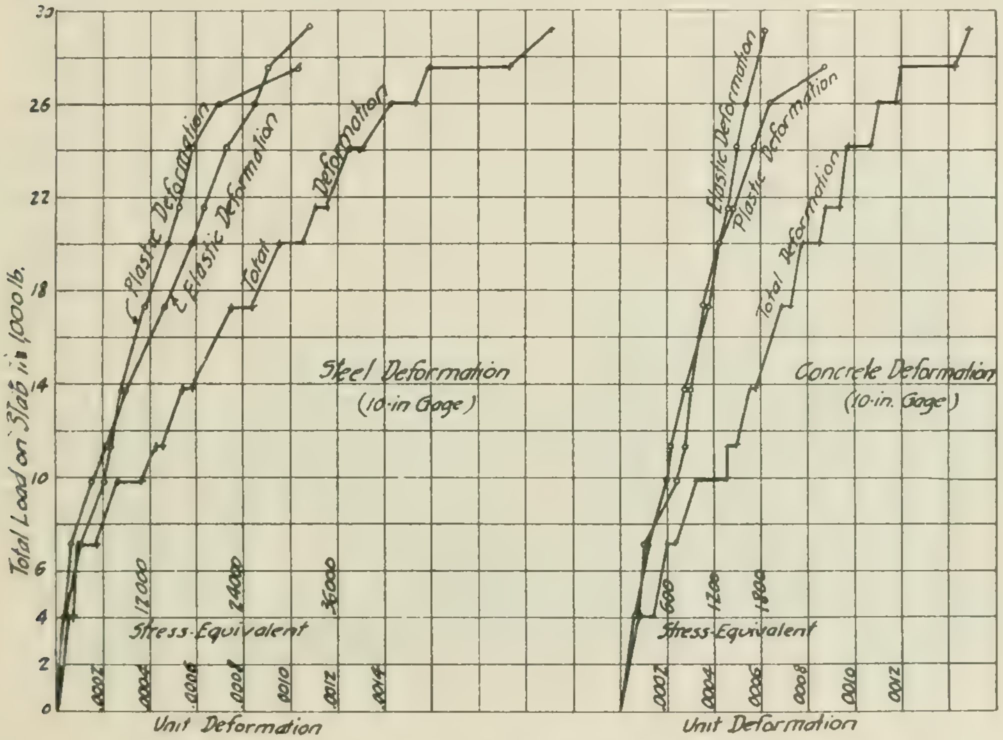


FIG. 7.—PLASTIC DEFORMATION IN CONCRETE AND STEEL.

CONCLUSIONS.

The conclusions formed by the writer from the results obtained on the temperature series are:

1. A 20° drop in temperature of a flat slab may very probably produce stresses in the steel and concrete equivalent to those obtained in the slab under load for which it was originally designed.

2. The importance of deformation and deflection due to temperature alone, and the wide variation obtained in some cases between various means of eliminating the temperature effect from the observations, leads the writer to believe that, wherever tests are made on a structure subject to a considerable temperature variation, a series of temperature readings, similar perhaps to those described in the foregoing paper, should be conducted

before the load test is begun in order to determine the law of variation due to temperature change for that particular structure.

The conclusions which the writer has formed through the study of plasticity deformation on slabs of a simple span and on flat slabs follow:

1. Panel deflection measurements should not be used as a characteristic of the behavior of a building unless these deflections are accompanied by sufficient plastic observations to eliminate from them plastic defect due to continued load.

2. Tests of structures should be so conducted as to indicate plastic deformation.

3. Loading of structures for test purposes should be carried on in regular intervals, and at regular periods of time between applications of load.

CONCRETE WORK ON THE BROOKLYN ARMY BASE.

By Arthur C. Tozzer.*

The Army Supply Base, constructed for the War Department at South Brooklyn, N. Y., provides approximately 4,000,000 sq. ft. of floor space in the warehouses for receiving, storing and shipping supplies for the United States Army.

The work comprises the following construction: Two eight-story and

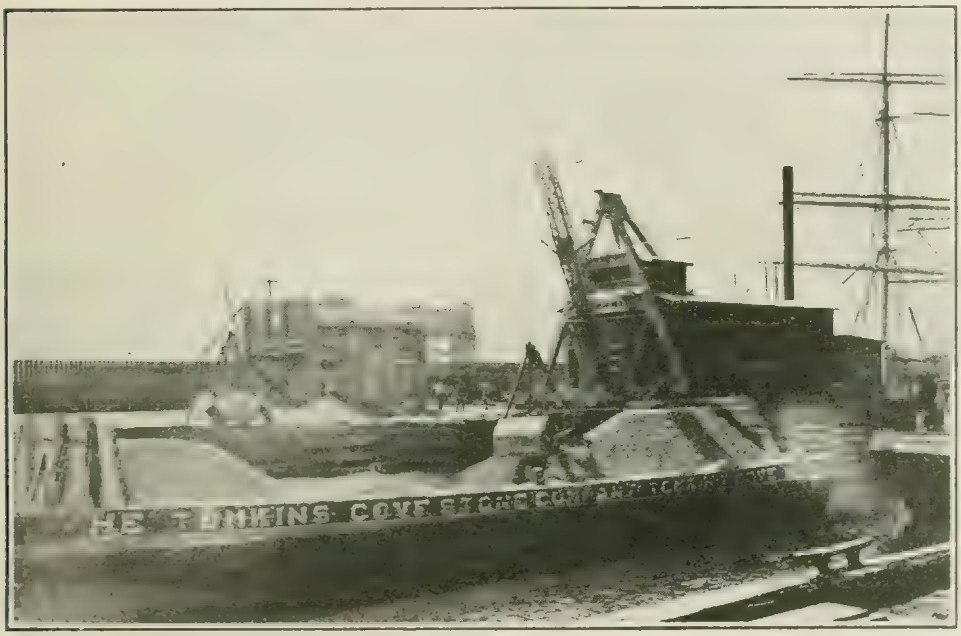


FIG. 1.—CLAM-SHELL BUCKET UNLOADING STONE.

basement reinforced-concrete warehouses, of two-way flat-slab construction —Building A being 200 x 980 ft., and Building B 306 x 980 ft. with an interior covered court 66 ft. wide; a boiler house, 93 x 138 ft.; a four-story and basement Administration Building, 70 x 200 ft.; three double-deck piers, 150 x about 1300 ft.; one open pier, 60 x 1300 ft.; and twenty miles of standard-gage track with accommodations for 1300 cars. The entire project covers an area of approximately 100 acres.

The warehouses, with the exception of a portion of Building B, have reinforced-concrete spread footings bearing on soil, those of the north and part of the center section of Building B bearing on Raymond concrete piles. Wooden piles are used for the support of the piers. The decks of the piers are of reinforced concrete. In order to prepare the site to receive the

*Executive Manager, Turner Construction Co., Army Supply Base, Brooklyn, N. Y.

buildings, land excavation amounting to, approximately, 600,000 cu. yd. was done by steam shovels. Three-fifths of this excavation was completed within two months.

The speed of construction of the entire project depended almost wholly on the facilities for the receiving and distribution of building materials. Fortunately, as is shown on the general plot plan (Fig. 3), part of the property acquired embraced the old railroad yard, and slips to the south, owned by the Brooklyn Rapid Transit Co. This slip allowed all concrete aggregate and cement to be received direct by water from the various banks and mills. The gravel supply of New York Harbor, prior to the starting of the Army Supply Base, had been allocated to other Government work, so that it was necessary to use, throughout the entire building work,

FIG. 2.—BATTERY OF CEMENT BELT CONVEYORS.

crushed trap rock from Hudson River quarries, this being graded material from 1½ to ¼ in. Sand and grits came from the banks adjacent to Huntington Bay, Long Island Sound. Four brands of cement were used, this cement being supplied from various Hudson River mills.

At the dock, barges containing concrete aggregate were unloaded by two large skid diggers, with clam-shell buckets of 5 and 3½ cu. yd. capacity, respectively (Fig. 1). Two boom derricks, each with 1½ cu. yd. buckets, cleaned up the scows after the larger machines had taken off the greater portion of the load. The buckets dropped the material directly into storage bins of about 20 cu. yd. capacity, from which it was loaded into 5-ton dump-bodied motor trucks which passed beneath the bins.

Cement in bags was received in covered barges, and rapidly unloaded

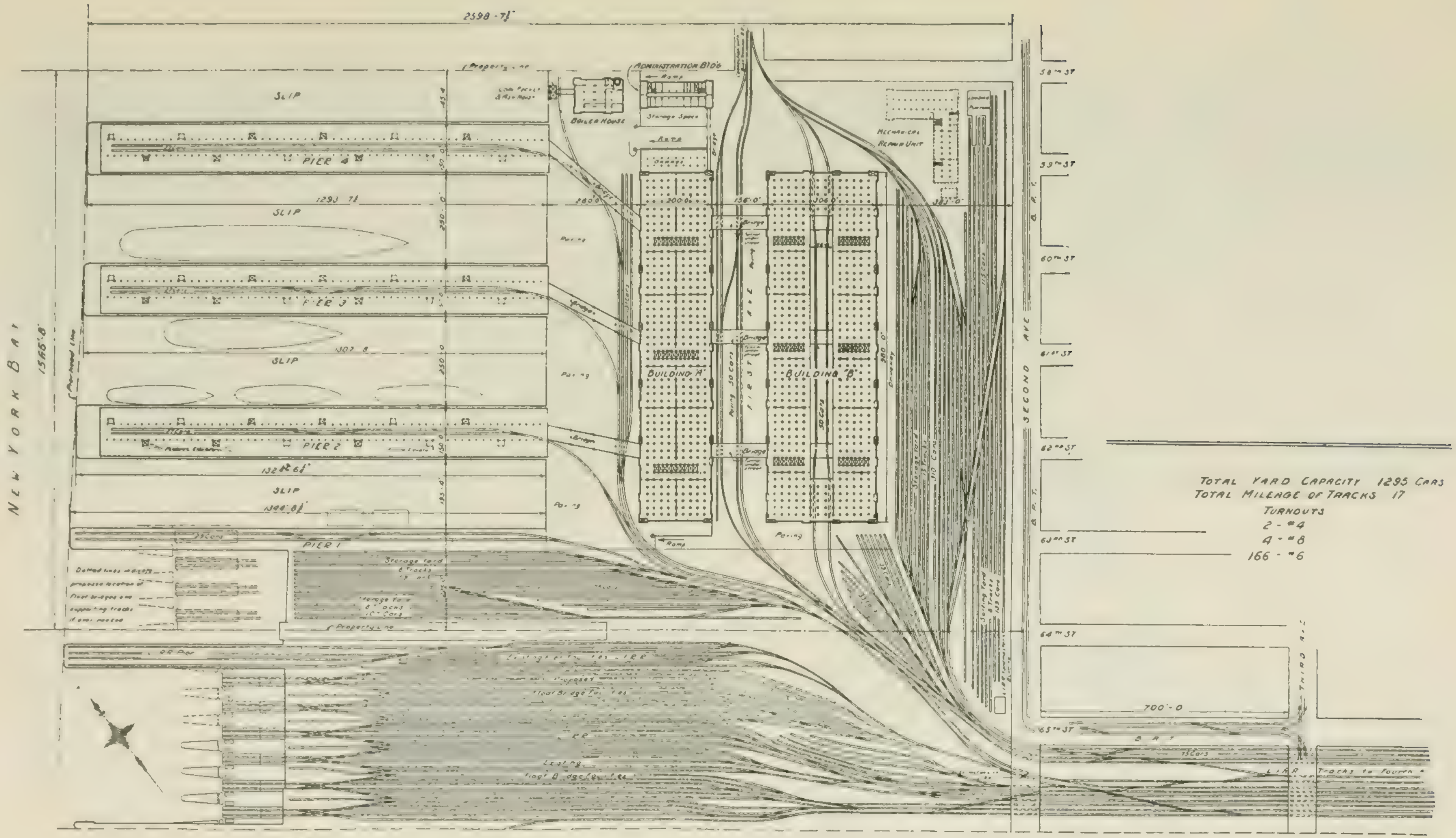
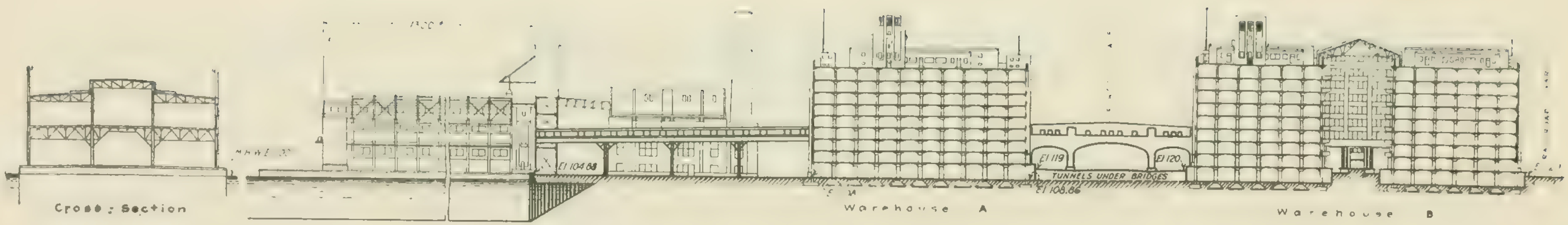


FIG. 3.—GENERAL PLOT PLAN AND CROSS-SECTION THROUGH BROOKLYN ARMY BASE.



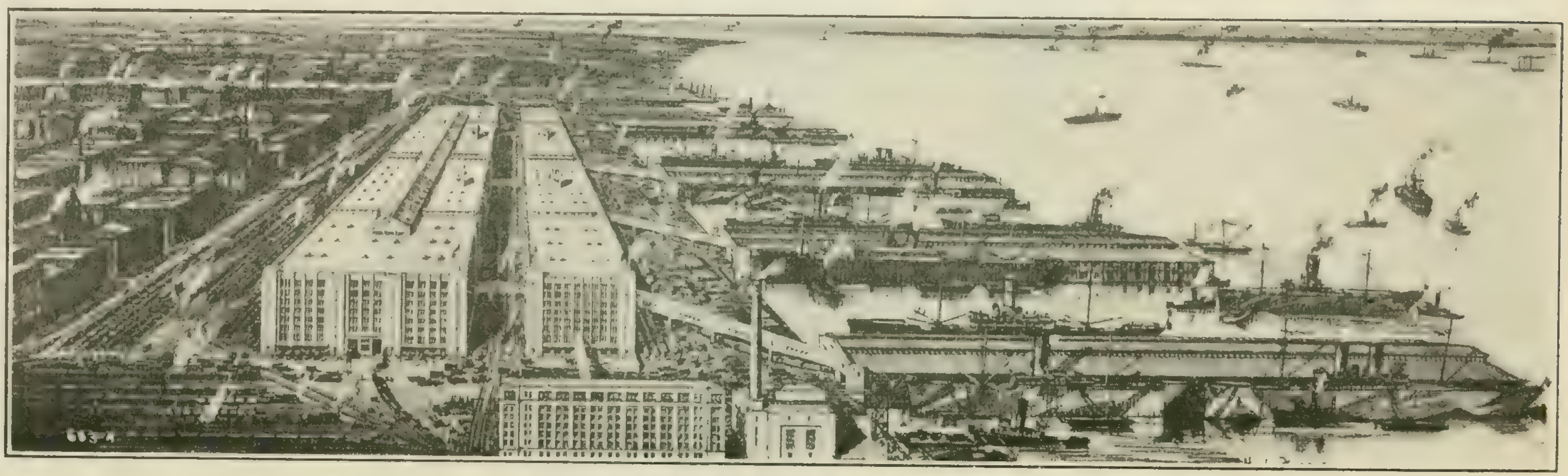


FIG. 4.—TRANSVERSE SECTION (ABOVE) AND ARCHITECT'S PERSPECTIVE (BELOW) OF BROOKLYN ARMY BASE.

from the boats by continuous belt conveyors built on the job. A battery of six of these machines (Fig. 2) was kept busy at the height of the construction season, unloading sufficient cement to supply the various plants.

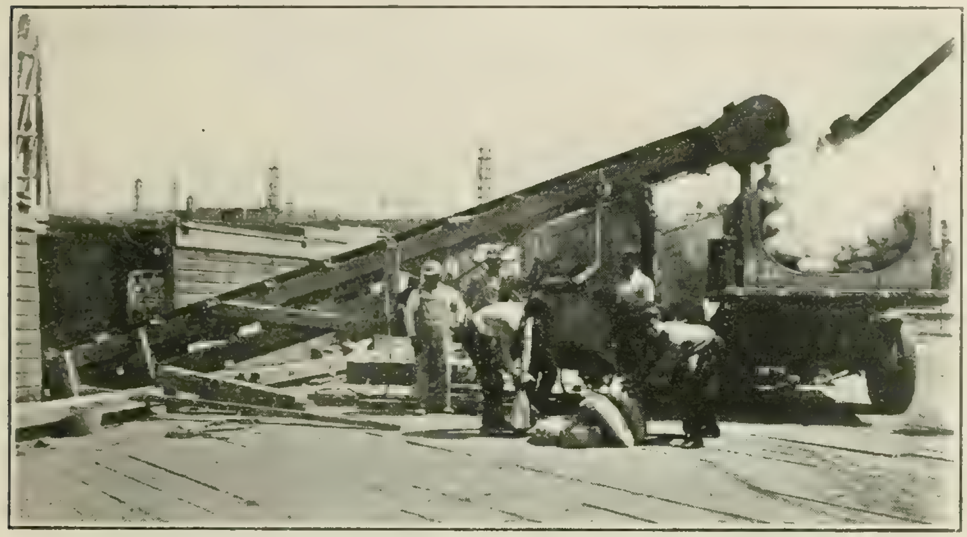


FIG. 5.—SINGLE CEMENT BELT CONVEYOR.

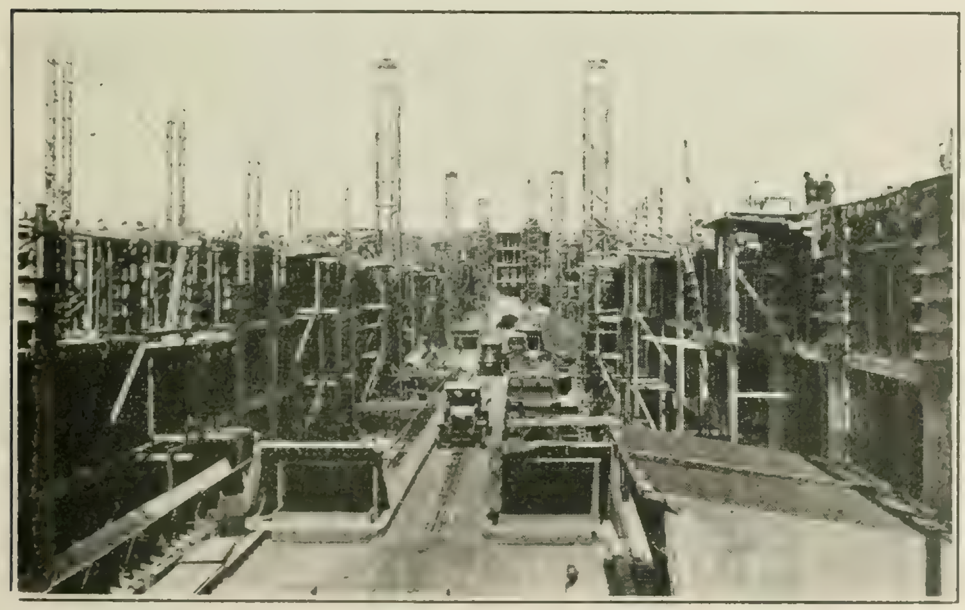


FIG. 6.—VIEW OF COURT, BUILDING B, SHOWING MATERIAL TRESTLE AND AREA OVER BINS.

Being substantially constructed, and supported on wheels (Fig. 5), these conveyors were easily moved from one location to another, and automatically adjusted themselves in the scows to the rise and fall of the tide. A 15-h.p. motor furnished power to the continuous belt on which the bags

were placed in the scows and conveyed to trucks. Cement was unloaded at the rate of 30,000 bags in ten hours. Six men were required to feed each belt on the scow, and one man to receive and arrange the bags on each truck.

The warehouses were served by timber trestles running along the street side of Building A, and through the court of Building B (Fig. 6).

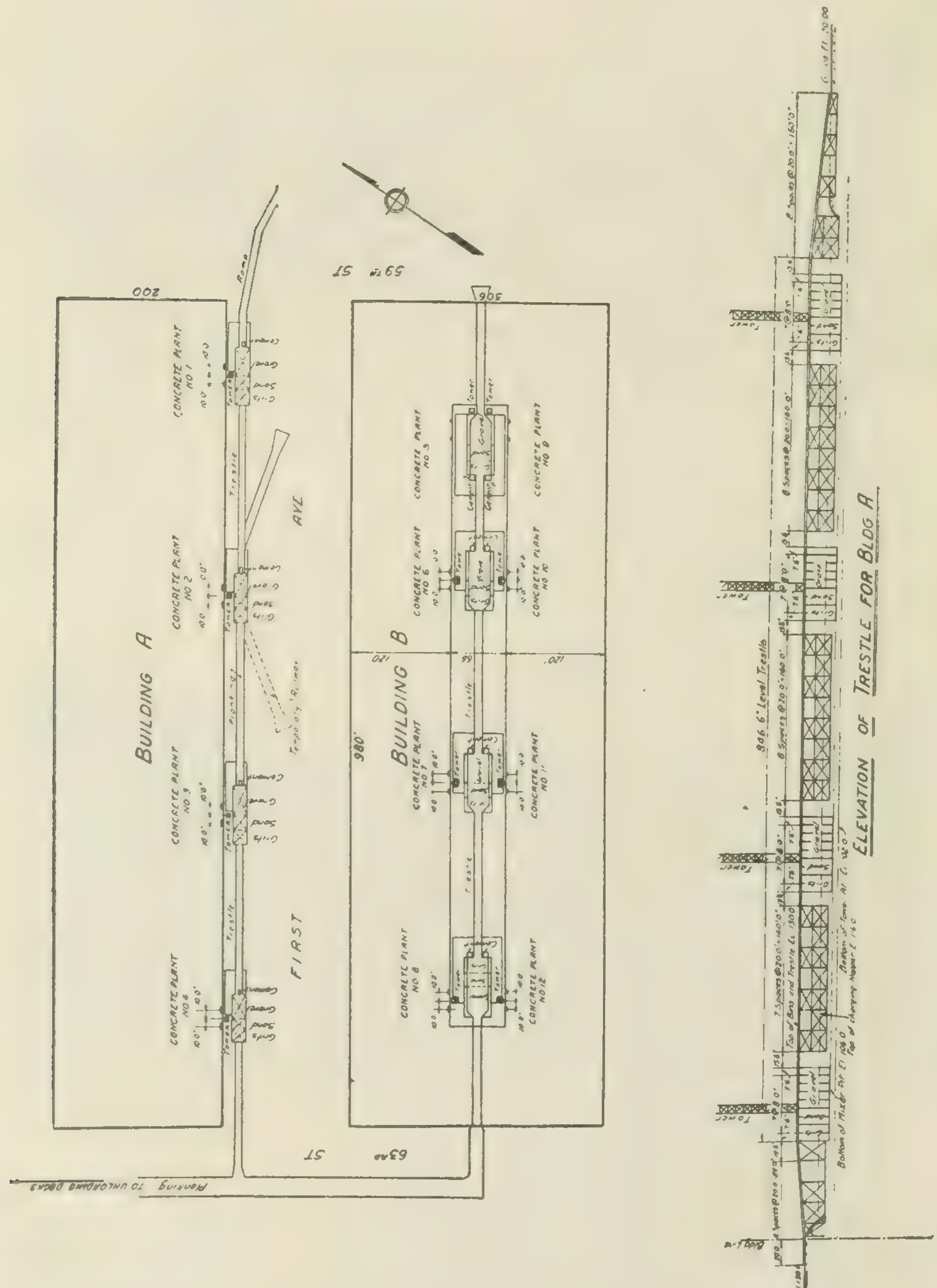


FIG. 7.—PLAN SHOWING TRESTLE AND ROUTE OF MATERIAL TRUCKS; ALSO ELEVATION OF TRESTLE.

These runways were about 25 ft. high and allowed ample room beneath for storage of the sand, stone and grits which were dumped directly through gridirons (made up of 4 x 10-in. planks on edge) into the bins. The bins beneath the grating were divided, and provided storage for 80 cu. yd. of grits, 80 cu. yd. of sand, and 160 cu. yd. of gravel, each.

The cement was dumped directly from the trucks into a chute connecting with a storage house of 4000 bags capacity.

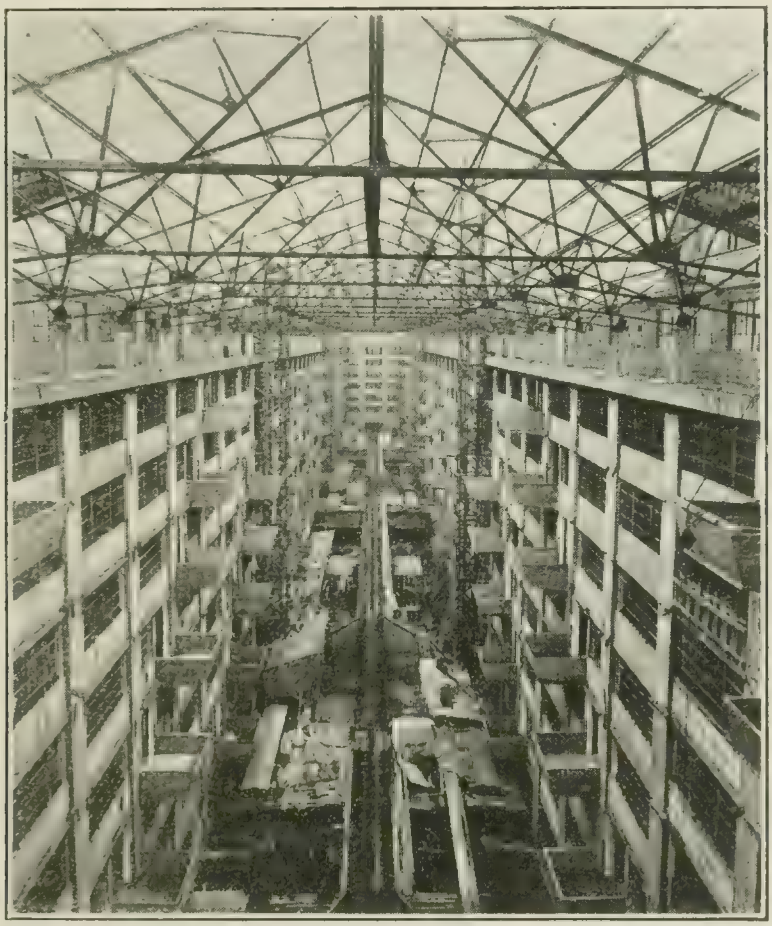


FIG. 8.—VIEW OF COURT, BUILDING B, SHOWING TRESTLE, BALCONIES AND SKYLIGHT.

In Building A four mixing plants and storage bins were installed, and in Building B eight, arranged on opposite sides of the interior court (Figs. 5 and 6.) At each point where a mixing plant was installed the trestle was widened out over the bins sufficiently to allow for dumping the material into any portion of the bin, and also to give room for other trucks to pass. From the bins the concrete material was fed directly through gates into 1¼-yd. side-dump cars, which were pushed by hand to the charging hopper of the mixers. Cement was dumped directly into the

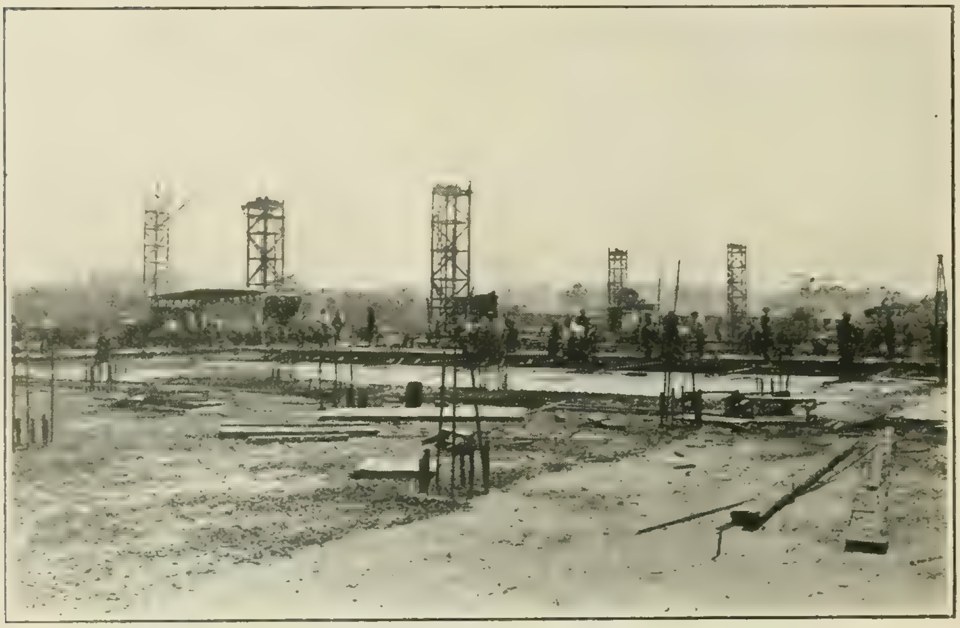


FIG. 9.—GENERAL VIEW OF TYPICAL FLOOR, SHOWING CONCRETE POURED, ROUGHENED, WHERE FINISH IS TO BE APPLIED LATER.

FIG. 10.—PROGRESS VIEW OF BUILDING A, TAKEN JULY 27, 1918—THIRD FLOOR STARTED.

charging hopper from bags. Each mixer had a capacity of one cubic yard of mixed concrete. All was 1 : 2 : 4 mixture, except for columns, which was 1 : 1½ : 3.

FIG. 11.—SAME AS FIG. 10, TAKEN AUG. 25, 1918.—SIXTH FLOOR STARTED.

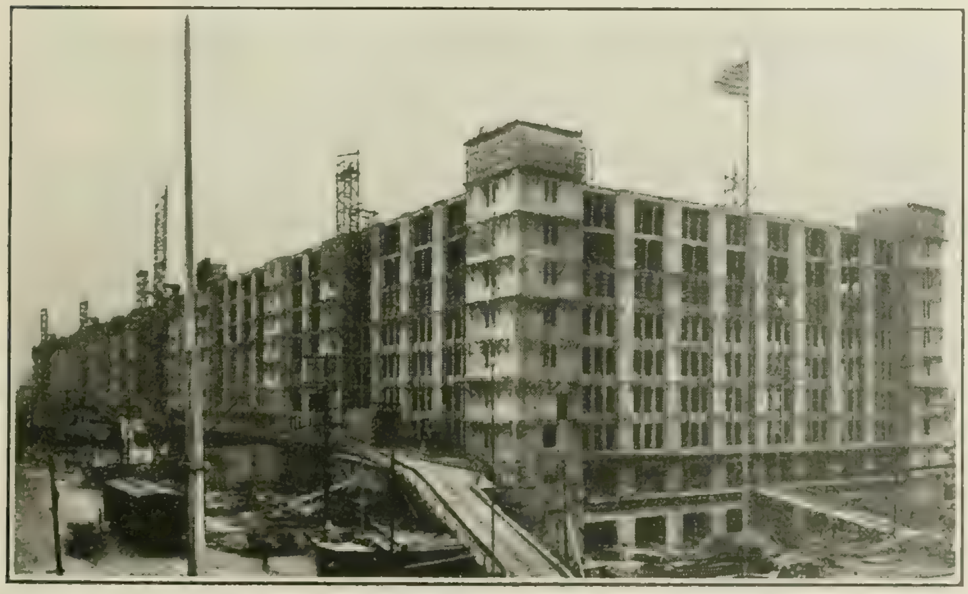
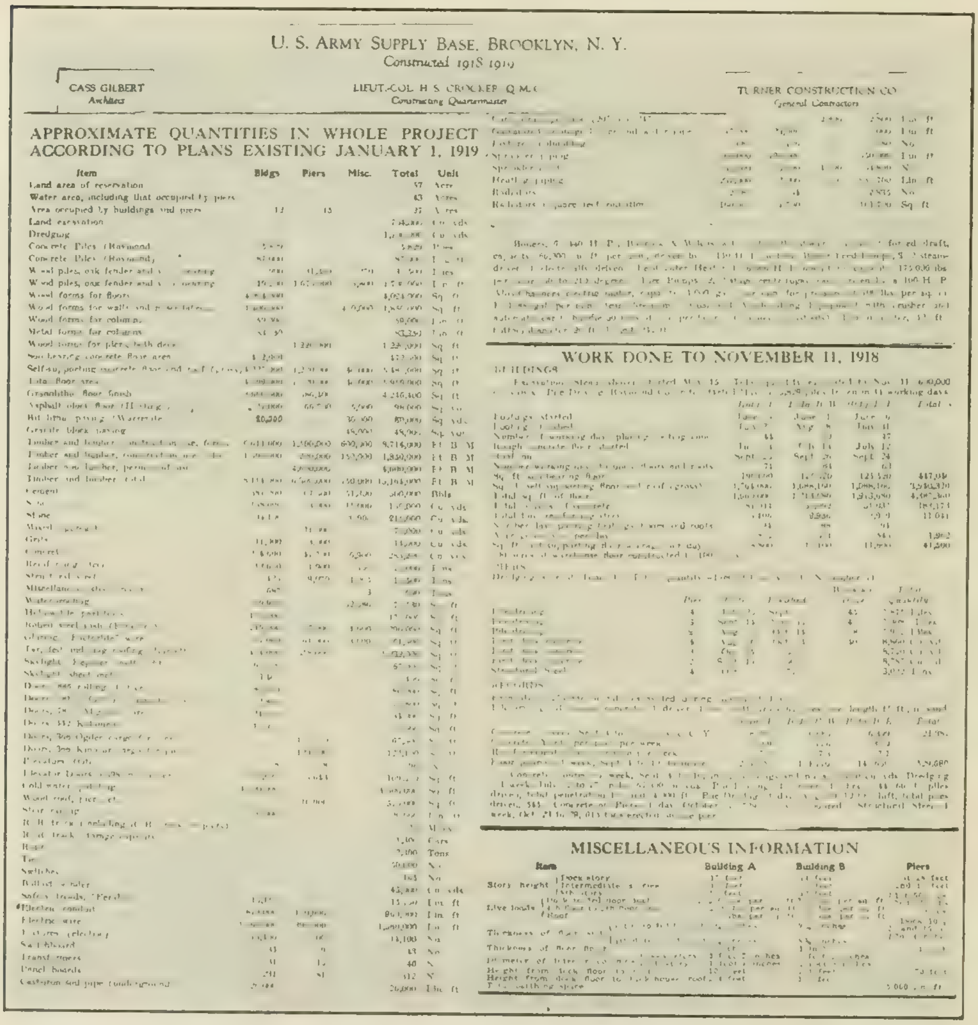


FIG. 12.—SAME AS FIG. 10, TAKEN SEPT. 23, 1918—ROOF COMPLETED.

The concrete was elevated to the various floors in the usual towers of timber construction, and deposited on the floors by two-wheeled carryalls of 6 cu. ft. capacity.

Grit was used for floor finish in a mixture of 1 cement, ½ sand, and 1½ grits. The slabs were left rough, and a finish 1 to 1¼ in. in thickness applied to the floors as soon as the posting for floor above was removed. Integral liquid hardener was specified to give a better wearing surface.

The appended cuts (Figs. 8, 9, 10, 11 and 12) show the progress made on the warehouses. The table of quantities (Fig. 13) shows, approxi-



U. S. ARMY SUPPLY BASE, BROOKLYN, N. Y.
Constructed 1918 1919

CASS GILBERT, *Architect* — LIEUT.-COL. H. S. CROCKER, Q. M. C., *Constructing Quartermaster* — TURNER CONSTRUCTION CO., *General Contractors*

APPROXIMATE QUANTITIES IN WHOLE PROJECT ACCORDING TO PLANS EXISTING JANUARY 1, 1919

Item	Bldgs	Piers	Misc.	Total	Unit
Land area of reservation				57	Acres
Water area, including that occupied by piers				43	Acres
Area occupied by buildings and piers	12	15		[illegible]	Acres
Land excavation				[illegible]	Cu. yds.
Dredging				[illegible]	Cu. yds.
Concrete Piles (Raymond)	[illegible]			[illegible]	[illegible]
Concrete Piles (Raymond)	[illegible]			[illegible]	[illegible]
Wood piles, oak fender and [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wood piles, oak fender and [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Lin. ft.
Wood forms for floors	[illegible]			4,024,000	Sq. ft.
Wood forms for walls and [illegible]	[illegible]		[illegible]	[illegible]	Sq. ft.
Wood forms for columns	[illegible]			50,000	Lin. ft.
Metal forms for columns	[illegible]			[illegible]	Lin. ft.
Wood forms for piers, both decks		1,320,000		1,320,000	Sq. ft.
[illegible] concrete floor area	[illegible]			[illegible]	Sq. ft.
Self-supporting concrete floor and roof [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Sq. ft.
Total floor area	[illegible]	[illegible]	[illegible]	[illegible]	Sq. ft.
Granolithic floor finish	[illegible]	[illegible]		4,246,400	Sq. ft.
Asphalt block floor [illegible]	[illegible]	[illegible]	[illegible]	96,000	Sq. [illegible]
Bitulithic paving (Warrenite)	20,200		30,400	[illegible]	Sq. yds.
Granite block paving			45,000	45,000	Sq. yds.
Timber and lumber [illegible]	[illegible]	1,500,000	600,000	9,714,000	Ft. B. M.
Timber and lumber, [illegible]	[illegible]	[illegible]	150,000	1,850,000	Ft. B. M.
Timber and lumber, permanent use		[illegible]		4,000,000	Ft. B. M.
Timber and lumber, total	[illegible]	[illegible]	[illegible]	15,164,000	Ft. B. M.
Cement	[illegible]	[illegible]	[illegible]	300,000	Bbls.
Sand	[illegible]	[illegible]	[illegible]	[illegible]	Cu. yds.
Stone	[illegible]		[illegible]	215,000	Cu. yds.
Mixed [illegible]		[illegible]		7,000	Cu. yds.
Grits	11,000	[illegible]		[illegible]	Cu. yds.
Concrete	[illegible]	[illegible]	[illegible]	[illegible]	Cu. yds.
Reinforcing steel	[illegible]	[illegible]	[illegible]	[illegible]	Tons
Structural steel	[illegible]	[illegible]	[illegible]	[illegible]	Tons
Miscellaneous [illegible]	[illegible]		3	[illegible]	Tons
Waterproofing	[illegible]		[illegible]	[illegible]	Sq. ft.
[illegible]	[illegible]			[illegible]	Sq. ft.
Rolled steel sash [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Sq. ft.
Glazing [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Sq. ft.
Tar, felt and slag roofing [illegible]	[illegible]	[illegible]		[illegible]	Sq. ft.
Skylight [illegible]	[illegible]			[illegible]	Sq. ft.
Skylight sheet [illegible]	[illegible]			[illegible]	[illegible]
Doors, [illegible] rolling [illegible]	[illegible]			[illegible]	Sq. ft.
Doors, [illegible]	[illegible]			[illegible]	[illegible]
Doors, [illegible]	[illegible]			[illegible]	Sq. ft.
Doors, [illegible] Kalamein	[illegible]			[illegible]	Sq. ft.
Doors, [illegible]		[illegible]		[illegible]	Sq. ft.
Doors, [illegible]		[illegible]		[illegible]	Sq. ft.
Elevators [illegible]	[illegible]	[illegible]		[illegible]	No.
Elevator towers [illegible]	[illegible]	[illegible]		[illegible]	Sq. ft.
Cold water [illegible]	[illegible]			[illegible]	Sq. ft.
Wood roof, piers [illegible]		[illegible]		[illegible]	Sq. ft.
Stair [illegible]	[illegible]			[illegible]	Lin. ft.
R. R. track [illegible]				[illegible]	Miles
R. R. track [illegible]				[illegible]	Cars
Rails				[illegible]	Tons
Ties				[illegible]	No.
Switches				163	No.
Ballast [illegible]				45,000	Cu. yds.
Safety treads, [illegible]	[illegible]			[illegible]	Lin. ft.
Electric conduit	[illegible]	[illegible]		[illegible]	Lin. ft.
Electric wire	[illegible]	[illegible]		1,000,000	Lin. ft.
Fixtures (electric)	[illegible]	[illegible]		14,100	No.
Switchboard	[illegible]	[illegible]		[illegible]	No.
Transformers	[illegible]	[illegible]		40	No.
Panel boards	[illegible]	[illegible]		312	No.
Cast-iron soil pipe (underground)	[illegible]			26,000	Lin. ft.

WORK DONE TO NOVEMBER 11, 1918

MISCELLANEOUS INFORMATION

Item	Building A	Building B	Piers
Story height	[illegible]	[illegible]	[illegible]
Live loads	[illegible]	[illegible]	[illegible]
Thickness of floor slab	[illegible]	[illegible]	[illegible]
Thickness of floor [illegible]	[illegible]	[illegible]	[illegible]
Diameter of [illegible]	[illegible]	[illegible]	[illegible]
Height from [illegible]	[illegible]	[illegible]	[illegible]
Height from deck floor to [illegible]	[illegible]	[illegible]	[illegible]

FIG. 13.—TABLE OF QUANTITIES; WORK DONE TO NOV. 11, 1918, AND MISCELLANEOUS INFORMATION.

mately, four millions square feet of self-supporting floors and roof concreted in 72 days, or, with foundations included, more than 100 acres of self-supporting floor in 100 days. The general quantities of the whole project are also shown.

Spruce lumber was received in train-load lots at lumber yard at Sheepshead Bay, some five miles from the Army Base site. At the lumber yard one complete set of floor forms for all of the buildings was constructed.

Approximately seven million feet of lumber were used on the forms for the buildings, the total lumber and timber used on the whole project being about twenty and a half millions feet. The storing of material at the

FIG. 14.—GENERAL SIDE VIEW OF BUILDING B, SHOWING ROOF HOUSES AND SKYLIGHT.

FIG. 15.—TYPICAL INTERIOR VIEW.

lumber yard made it possible for one gang, with a complete sawmill outfit, to fabricate all of one definite kind of forms, so that maximum speed and minimum cost was possible. Some ten portable saw tables were used in this

yard. Forms after being made were transferred to the building site by the Long Island R. R., and also by motor trucks, and were delivered as required at the points where they were to be used.

All reinforcing steel was received directly from the mills in cars, and unloaded into storage piles. As most of the buildings are of flat-slab construction, very little fabrication was required for floor steel. The spirals for interior columns were received wound to proper diameter, and were made up on the site. All steel, as required, was transferred from the

FIG. 16.—PIER 4, SHOWING FORM WORK FOR SECOND DECK.

storage yard to the various parts of the work on narrow-gage push cars.

The handling of the concrete work on the boiler house and the Administration Building was similar to that on the warehouses, excepting for minor differences in the plant layout.

A maximum of fifty-two 5-ton dump-body motor trucks were in use on the stevedoring of concrete material, and about one hundred on excavation.

The concrete work on the piers consisted of a first deck, resting directly on the wooden pile caps, and a second deck supported on structural steel frame (Fig. 16). The first decks were concreted from two floating mixing plants, which were provided with clam-shell buckets for unloading the

gravel mixture from the scows into the mixer hopper. The second decks were beam and girder construction, with monolithic finish, and were concreted partly from floating plants and partly from stationary plants. It was found much more economical to use the gravel mixture on the piers than to use the unmixed material, on account of the fact that less storage space was required. At the time the pier work was under way, the building work was well advanced; and, moreover, the demand for gravel on Government work had somewhat decreased. Hastings' asphalt blocks some of the truck aisles in the warehouses.

The entire area of the property, exclusive of that occupied by the buildings and railroad yards, is paved. The streets, and portions subjected to heaviest traffic, have a 6-in. concrete sub-base with granite blocks. The farm, or area between Warehouse A and the piers, has an 8-in. concrete base, with 2-in. asphalt wearing surface.

The total amount of concrete work on the project is, approximately, 288,000 cu. yd., divided as follows:

Buildings	215,000	cu. yd.
Piers	40,000	"
Paving	15,000	"
Miscellaneous	18,000	"
Total	288,000	"

The work on excavation started May 15, 1918, and the roofs of all the warehouses were completed on Sept. 26, 1918. The entire project is now (June, 1919) substantially completed, and is in use by the Storage Division of the Army.

In order to obtain maximum efficiency from the men working under war conditions, an intensive propaganda of stimulation was carried on. A weekly job publication, known as the *Army Base Mixer*, gave pictures of the men and the work, and the news of what was going on about the work. Keen competition resulted, due to the inauguration of a complete system of records showing the individual efficiency on the various classes of work. Every two weeks the section showing the greatest efficiency per man, together with the general accomplishments, was awarded the Honor Flag, which symbolized that this section had made the best showing. The friendly rivalry among the gangs on the various portions of the work spurred the men on to greater efficiency.

After the signing of the armistice there was a very general let down in all war work. A decrease of, approximately, 25 per cent in the labor forces followed the War Department's putting the work on an eight-hour basis. This was followed by continual strikes in the various trades, which has delayed the final completion of the work.

The entire project offered, in almost every phase of construction work, many problems, the magnitude of which has never been met, so that the Brooklyn Army Supply Base stands as a monument to the intention of the United States to prosecute the war to a victorious conclusion.

DISCUSSION.

Paper Presented by Mr. E. J. Moore and Mr. A. W. Stephens.

Mr. Hardison. Mr. R. M. Hardison.—How many expansion joints were provided in each building?

Mr. Moore. Mr. E. J. Moore.—There were two expansion joints in the total length of 980 ft., dividing each building into three sections.

A Member. A Member.—What is the detail of the expansion joints? Is the building cut through completely?

Mr. Stephens. Mr. A. W. Stephens.—The building is cut through completely from the roof to the top of the footings. The footings are common for the two columns which adjoin each other.

Mr. Turner. Mr. H. C. Turner.—It may be interesting to say that those two warehouses are filled with supplies which had been purchased to ship to France. Now that the war is over and we are not shipping them across the ocean, the two warehouses are full of everything. We do not know when they will be unloaded. Two of the piers are in operation for unloading.

Mr. Hatt. Mr. W. K. Hatt.—Are the flat-slab floors designed according to the New York or Chicago code.

Mr. Stephens. Mr. A. W. Stephens.—They approach more nearly to the Chicago code.

Mr. Wight. Mr. F. C. Wight.—It is worth while to note here that this base, together with the similar ones at Boston and at New Orleans—all three are of somewhat similar design and layout—make together probably the largest aggregation of concrete buildings that have ever been built at one time. All were built at remarkably high speed. The contribution of concrete to this problem of port and terminal handling is, I think, one of the greatest developments of the war. It can only be hoped that the lessons learned in the design and the lessons that will undoubtedly be learned in the operation of these three big army bases will serve materially to develop a hitherto undeveloped science in this country—that is, the science of loading and unloading ships economically.

CONCRETE RAILWAY TRACK SUPPORT.

A. C. Irwin.*

Practically all expenditures that have been devoted to the improvement of railroad facilities and equipment may be traced to the necessity of obtaining greater traffic concentration. The tractive power and speed of locomotives have been enormously increased in order that longer and heavier trains may be used. The carrying capacity of cars has been increased manyfold so that a greater weight may be carried in a smaller length of train. Bridges have been replaced with stronger ones and terminal facilities have been enlarged and made more efficient through the introduction of special appliances for handling freight. Automatic block signals have been introduced so that trains may follow each other in quick succession. Many millions of dollars have been spent in reducing ruling gradients so that longer and heavier trains may be employed and so that the time required to pass between given points may be reduced. All of these improvements are centered around the one proposition of gaining the greatest possible concentration of use of track.

It is an almost unbelievable fact that the most fundamental factor concerned with greatly increased density of traffic, namely, the track itself, has been practically neglected in so far as engineering design is concerned. The obvious solution of any given problem is too often followed. Obviously, if heavier trains are desired, more powerful motive power must be furnished and therefore stronger bridges, etc. If trains are to be run closer together some system must be devised for protecting them which would not require an operator every three-fourths mile or so, hence the block signal systems. All these are obvious ways of obtaining greater use of track and are necessary to the accomplishment of the desired end. However, they have been carried out apparently with forgetfulness of the effect of heavy locomotives and cars on the track itself and the maintenance thereof and without any serious attempt to evolve a track construction adequate to the demands upon it.

O. E. Selby said in Bulletin No. 80 of the American Railway Engineering and Maintenance of Way Association: "Railroad track has grown in strength as heavier loads have made increased strength necessary, but such growth has been entirely along empirical lines and not one single detail of track superstructure bears marks of engineering design." It is true that heavier rails have been used, but the extra strength afforded by these rails has not at all been in proportion to the increased loads which they are called upon to carry. A greater number of ties have been used per rail; that is, ties have been spaced closer together, thus affording some increase in bearing area on the ballast, but this increase has lagged far

*Portland Cenemt Association, Chicago, Ill.

behind the increase in wheel loads. A greater thickness of ballast has been used, but since there is nothing to retain this ballast in any given position it spreads out and is driven into the subgrade by the heavy loads.

That the ways and means above referred to of obtaining greater traffic concentration have not been considered in all their aspects is evidenced by the increase in maintenance costs. It is a well-known fact that any machine, structure or living thing, for that matter, wears out with astonishing rapidity after a certain point of strain is reached and passed. The limit of the supporting power of the ordinary track construction at which the track could be economically maintained was long since passed and the result is that we are now endeavoring to keep in first-class condition a structure which is being continually stressed beyond the point where such stress can often be repeated. This accounts in a large measure for the fact that increased cost in maintenance of track has largely nullified the advantage aimed at through the expenditure of millions of dollars on motive power, grade reduction, block signaling, improved terminal facilities, etc.

Let us examine a little more specifically some of the causes of these high maintenance costs.

HIGH RAIL STRESSES.

The progress report of the Special Committee on Stresses in Railroad Track of the A. R. E. A. for March, 1918 (Bulletin No. 205, A. R. E. F.), points out that the "Rail stresses differ markedly according to the condition of the track—freshly tamped track giving smaller stresses than track which has been subjected to the action of traffic for a considerable time after receiving a general surfacing." This report also shows that what is described as "track in excellent condition" gives rail stresses something like 6,000 lb. per sq. in. more for freshly-tramped track. The evident conclusions from these facts are that track in so-called "excellent condition" is subject to deflection that produces high rail stresses and that what we are in the habit of considering as well maintained track is in reality a track condition far from ideal. In fact, maintenance such as would prevent large deflection is wholly impracticable if not impossible. Ties cannot be tamped after the passage of every few trains, and even could such work be done the deflection allowed would still be too great to prevent high rail stresses.

We are not here concerned with the controversy as to whether more rail breakage is due to overstressing or to defects, but the incontrovertible fact is shown by the tests of this report and the record of other investigations that rail stresses as a result of bending due to insufficient support, are very great. Stresses as high as 40,000 lb. per sq. in. were found on cinder ballasted track with 56-lb. rails carrying a switching locomotive. Stresses of 25,000 lb. per sq. in. were found in 85-lb. rails with 12 in. of stone ballast with track in "excellent condition," and it must be remem-

bered that the rail stress is reversed many times with the passage of every train.

A tacit admission of the high, not to say dangerously high, stresses in rails in heavy high-speed traffic track is found in the general pracctice of replacing such rails with new ones in a few years at most regardless of the apparent condition of the rails. Evidently even a slight decrease in the new rail section due to wear, together with such internal weaknesses as develop in the rail under traffic, are considered to have sufficiently cut down the slim "factor of safety" as to warrant its replacement. Furthermore, track in "excellent condition" is not the kind of track that is resurfaced or tamped and before it is necessary to do this the condition of the track has deteriorated with consequent higher rail stresses and resulting rail breakage. It costs to take up rails, transport them and relay them in less exacting service, and replace them with new ones. In addition, damage to ties results and traffic is more or less interfered with. If rails were not called on to function as girders, but merely as wearing surfaces or contacts, much of this loss would be saved. Lighter rails could also be used.

THE RAIL-FASTENING PROBLEM.

While it is admitted that the rail fastening is a "burning question" in track design, it may be seen that the difficulty of devising an ideal fastening lies largely in the type of construction in use. Track rails are at least partially continuous and have numerous, but not fixed, supports. Whenever positive deflection occurs there must, therefore, be negative deflection, and a negative reaction of considerable amount results. Rail deflection, first one way and then another, rocks the ties, producing rail cutting and, together with the uplift due to negative bending, loosens and pulls the spikes. Plugging holes and respiking is the only alternative followed. This is not only costly in itself but contributes materially to the deterioration of the ties. Little serious attempt has been made in any manner having a logical chance of success to remedy the difficulty of a suitable rail fastening. The same deficient type of sub-construction, allowing large deflection, and the same relatively soft and impermanent supporting material is used. Rails having a continuous support would suffer practically no negative bending and consequently no negative reaction would be produced. This would simplify the rail-fastening device.

RAILWAY TIES.

Wooden ties, once plentiful and cheap, are now costly and sometimes difficult to obtain at all. This is especially true of hardwood ties such as are best suited to high-speed heavy traffic track. Great increase in the number and amount of wheel loads has naturally brought about more rapid depreciation of ties. In such track the ties are being stressed far beyond what would be allowed in buildings or bridges. They are, moreover, subjected to weakening from rail or tie plate cutting and

repeated spiking as well as abrasing and chipping from the frequent tamping required. As a matter of fact tie renewals are largely due to such damage, and this increases with traffic.

BALLAST.

In connection with Federal Valuation of Railroads, investigations were made to determine the depth of ballast under track ties. It was found, in general, that the ballast had been pounded and pushed into the subgrade to a surprising etxent, reaching, in places, more than 5 ft. This, together with experiments and experience, proves conclusively that the usual thickness of ballast under "first-class" track does not distribute the load uniformly to the subgrade, that large quantities of ballast are pounded and pushed into the subgrade, that dirt is worked up into the ballast, requiring much labor, expense for cleaning, and that the ballast item of track maintenance has greatly increased. This is to be expected. The load which the ballast brings to the subgrade is partially concentrated, and the ballast is made up of individual particles having very little cohesion and hence little value in distributing load over the subgrade. The shape of the ballast particles is, moreover, suited to penetration of the soft material in the subgrade. The heavy loads now in use have passed what may be termed the elastic resisting power of the subgrade to the particles of the ballast and rapid loss results.

FUNDAMENTALS OF TRACK DESIGN.

It should be apparent that the fundamental difficulty in obtaining a satisfactory track condition with the present type is in the type itself. It is not practicable with individual supports having no connection with each other longitudinally, excepting through the rails, resting on material possessing little or no cohesion and hence little ability to distribute loads uniformly to the subgrade, to obtain a structure that will be free from large deflections under heavy loads or which will maintain its position for any length of time without constant attention. In the light of what seems to us to be fundamental defects in the present type of track construction let us announce what we understand to be the fundamental requirements. This involves a track foundation having sufficient longitudinal and transverse strength to distribute the loads to a well-compacted subgrade so as not to exceed the bearing power of the subgrade, surmounted by rails held firmly in place in such a way as to be easily adjusted or replaced. With this sort of construction it would only be necessary to provide a smooth, hard, wear-resistive contact rail which would be supported throughout all its length. Modern track construction is a conglomeration of these ideas. The rail is at once a contact or wearing surface and a girder acting to distribute the load to the ties. Further, instead of the ties affording a construction having sufficient longitudinal and transverse strength to distribute the load uniformly to the ballast they are given a spacing so great

that each acts independent of the other excepting as affected by the strength of the rail and are of such size, strength and material as to be easily deflected.

TYPES OF CONSTRUCTION PROPOSED.

The ideal track construction will be had only when we have approached as closely as practicable to the above fundamental conditions. Types of construction designed to approach the fulfilment of these requirements have occasionally been proposed. Three general types of such designs are noted as follows:

First: Longitudinal stringers under the rails.

Second: A slab having some longitudinal and transverse strength laid directly on the subgrade and carrying ballast and ties.

Third: A slab having longitudinal and transverse strength laid directly on the subgrade and carrying the rails either directly on the slab or with blocks or longitudinal stringers between the rails and slab. In this type no ballast is used.

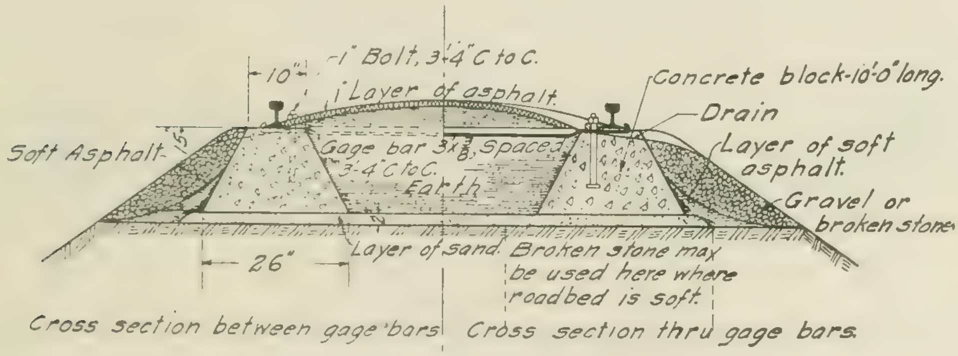


FIG. 1.—DESIGN FOR A PERMANENT WAY FOR STEAM RAILWAYS, SUGGESTED IN "ENGINEERING NEWS," JAN. 5, 1899.

In the early days of steam railways the first type of construction was used. The old strap-iron track really represented the idea of a longitudinal support for the rail or wearing surface proper. The longitudinal wooden stringer to which the strap was connected was expected through beam action to carry the loads and the strap merely to form a hard, smooth surface on which the wheels should run. The failure of this type of construction to survive was due to the use of insufficient and improper materials, together with poorly worked-out details.

The Jan. 5, 1899, *Engineering News* published an article by J. W. Schaub entitled, "A Design for a Permanent Track for Steam Railways," and in this number a staff article also appeared on the subject. The design proposed in the staff article is presented herewith as Fig. 1. The rails were laid directly on the concrete and held to gage by 1-in. rods on 18-in. centers and embedded in the portion of the concrete between the rails.

It will be noted that this design follows rather closely the original idea

of having a separate longitudinal stringer under each rail to furnish the required strength. The objection, however, to this design is found in the fact that these two stringers are not rigidly connected transversely so that they do not act together and the load is not distributed over the full width of the subgrade. An advantage claimed for this type is that by carrying the longitudinal girder below frost line, all track heaving would probably be prevented. The text of this article discusses in an interesting way some objections usually raised to solid track support.

"Doubtless the very first objection which will be brought against this design will be that it is lacking in elasticity. The story has been retold time and again in engineering literature how on some of the first railways ever constructed the rails were laid directly upon stone blocks, and it was found that the vibration due to the lack of elasticity in these supports brought about the rapid wear and deterioration of the rolling stock. From constant repetition of this story can be traced the belief that elasticity is a necessary element in a railway roadbed. Let us see how much there is in

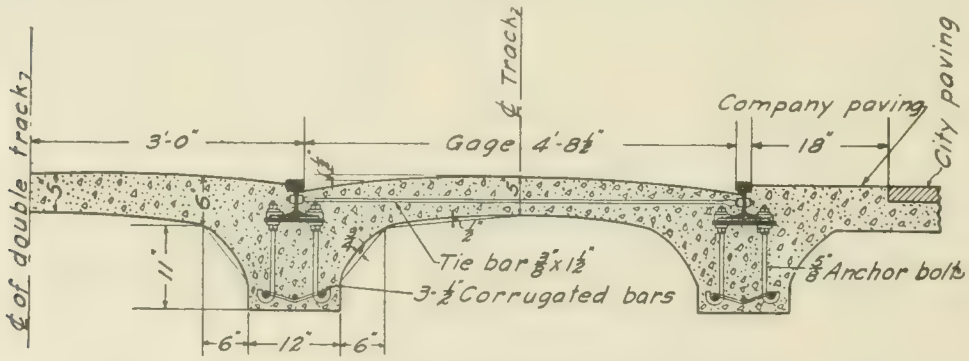


FIG. 2.—CONCRETE STREET RAILWAY TRACK CONSTRUCTION AT ANDERSON, N. C.

it. In the first place, it is easy to see that an elastic rail support was far more necessary with the rough cast-iron or wrought-iron rails of the '30's than it is with the wonderfully perfect steel rails of the present day. The fact is, of course, that the need for elasticity is in proportion to the amount of shock which is to be taken care of. One can easily understand how those rough old rails, with their frequent joints, resting on granite blocks not too smoothly dressed, may have been rapidly pounded out under traffic; but it does not follow at all that modern steel rails secured to a smooth and unyielding concrete foundation would suffer as did those early rails, or would cause wear and deterioration of the rolling stock.

"As it happens, moreover, we are not obliged to go so far back as the '30's for examples of rails resting directly on masonry supports. As many of our readers are aware, the latest and most successful system of street railway track construction dispenses entirely with ties and places the rail directly upon a longitudinal beam of concrete.

"With this system of construction, according to our best information, experience shows a smoother riding track and less wear of cars and machinery than with rails laid on cross-ties in the ordinary fashion. Here, it seems to us, is sufficient precedent for what we may call the 'concrete longitudinal system of track construction to secure for it at least a fair hearing. If modern electric cars, with their heavy motors suspended from the axles, can run over rails resting directly on a concrete base, it is at

least probable that steam railway trans can do the same; for it is a well-known fact that a much heavier track construction is required to stand up under electric car service than under the traffic of a trunk line railway."

Fig. 2 shows a type of track construction designed by E. R. Horton, Jr., and used in Anderson, Greenville and Charlotte, S. C., for street-car track construction. This is in reality the longitudinal girder principle. The rails are embedded in the concrete and rest on a continuous plate. Rail clips and hook bolt anchors are used, as shown in the drawing.

Recent inquiry in regard to the condition of this track, after four years' service, indicated that the construction has been found satisfactory in every particular, that vibration has not caused deterioration of the concrete and that even the paving between the tracks is in good condition. To adapt this design to steam railroad requirements it would only be necessary to deepen the girders and widen them out at the base for additional

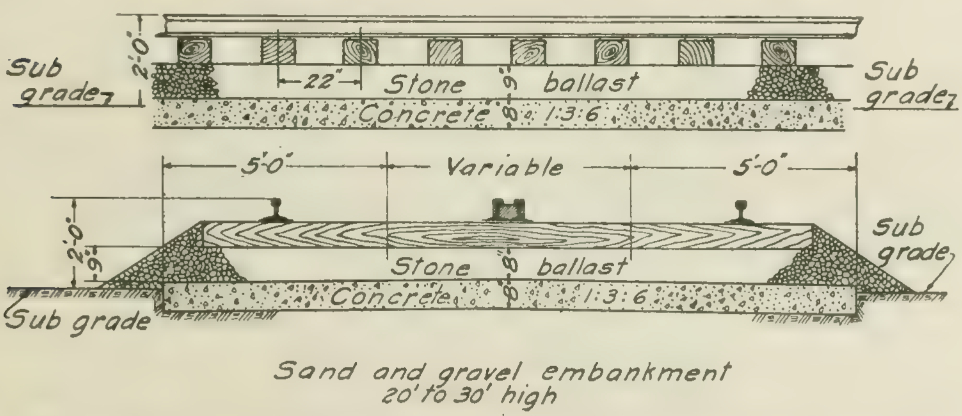


FIG. 3.- SECTION OF CONCRETE SLAB, JAMAICA IMPROVEMENT, LONG ISLAND R. R., BUILT 1912-1913.

bearing area and to change the paving in between tracks to diaphragms at intervals below the base of rails.

Examples of general Type 2, involving a concrete slab carrying ballast in which ordinary track ties are embedded, is shown by Fig. 3. This construction was placed under forty-nine crossings, switches and slips on the Long Island R. R. at Jamaica, N. Y. The work was put in during the winter of 1912-13 on embankments composed of sand averaging 20 ft. high. Portions of the slabs were placed without allowing time for the embankment to settle, and the traffic over these slabs has been extremely heavy from the day it was put in, running as high as 1,300 train movements per day. After three and one-half years, the General Manager of the Long Island Railroad, J. A. McCrea, reported that there had been practically no maintenance on the track. Mr. McCrea goes on to say that the great advantage of the slab is that the bearing surface on the natural ground is increased about three times over the usual method, without taking into consideration its continuity, and that examination of these experimental slabs showed that there were no cracks, at least in the vicinity of the

points of examination. It will be noted that these slabs are not reinforced and that they are only 8 in. thick. It should also be noted that they have been subjected to the pounding that occurs at crossings and frogs. There can, therefore, be no question of its practicability under main line track, but where so used it would seem appropriate to construct curbs at the ends of the slabs to hold the ballast from spreading, and to reinforce the slabs.

It has become very common practice to use reinforced-concrete slabs to form the decks of deck-girder railroad bridges. These slabs are usually about 1 ft. thick, 5 ft. long and span the distance between girders. The ballast is retained by curbs at the ends of the slabs. The testimony of railroad officials in regard to this construction is that when the ties are once embedded they stay in place and that maintenance is very greatly reduced.

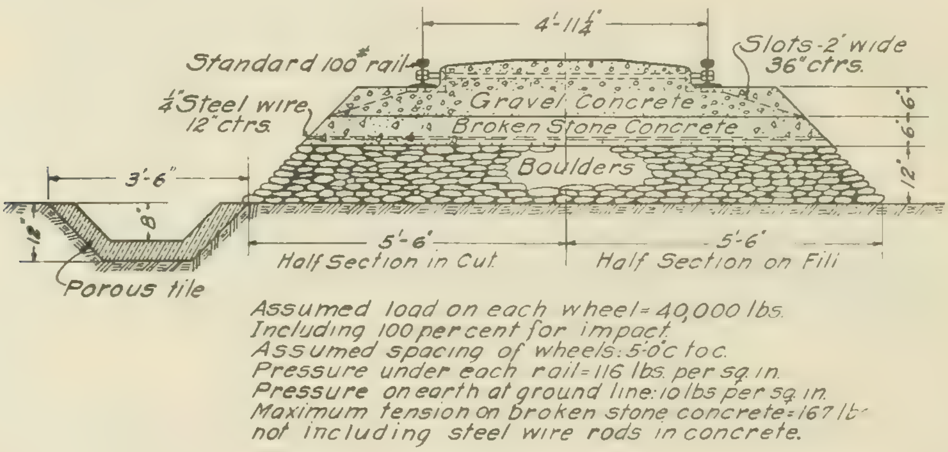


FIG. 4.—CONCRETE RAILROAD TRACK CONSTRUCTION PROPOSED BY J. W. SCHAUB, 1899.

Fig. 4 shows a design for a concrete railroad track proposed by J. W. Schaub in 1899. It will be noted that Mr. Schaub proposed to lay 100-lb. rails directly on the concrete. By specifying such a heavy rail section Mr. Schaub did not take full advantage of the continuous support afforded by the concrete, thus relieving the rail of practically all bending stresses. It is also clear that the 12 in. of boulders called for are entirely unnecessary.

In 1905 Mr. Schaub proposed a modification of his previous design. His later design is shown by Fig. 5. In this design a longitudinal wood stringer between the rail and the concrete is provided. This is evidently a concession to the rather prevalent notion that some elastic medium must exist between a track rail and a solid base. It also affords an opportunity to use the ordinary type of rail fastening and to replace or adjust the rails with facility. The 18 in. of rubble concrete shown is unnecessary. All stress requirements will be provided by making the slab 12 in. thick and resting it directly on a well-compacted subgrade.

The later Bergen Hill Tunnel of the D., L. & W. R. R. was completed in 1909. The tracks in this tunnel are carried on reinforced-concrete slabs, details of which are shown on Fig. 6. This construction was designed by

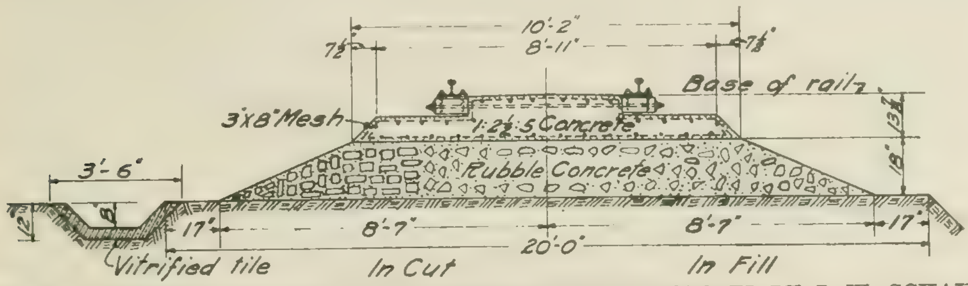


FIG. 5.—CONCRETE RAILWAY TRACK CONSTRUCTION PROPOSED BY J. W. SCHAUB, 1905.

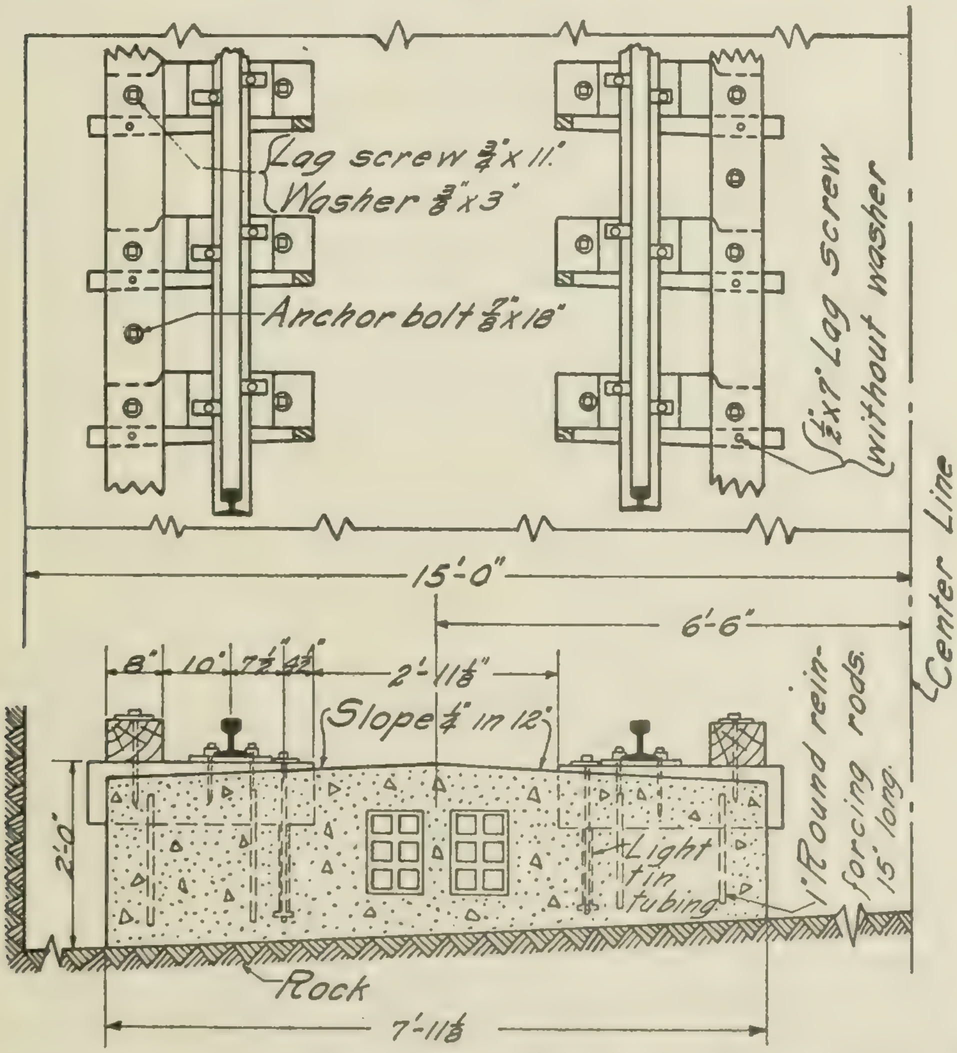


FIG. 6.—CONCRETE TRACK CONSTRUCTION PROPOSED BY LINCOLN BUSH AND USED IN BERGEN HILL TUNNEL, D., L. & W. R. R.

Lincoln Bush, who was then Chief Engineer of the D., L. & W. R. R. Under date of Oct. 3, 1916, Mr. Bush said as follows in regard to this construction:

"It was completed and turned over for service in February, 1909. The roadbed in the new tunnel has stood up exceedingly well and none of the creosoted tie blocks have been renewed so far as I have learned on recent inquiry.

"My idea in getting out this design was if railroad track could be made perfectly rigid and unyielding that there would be no pounding or unusual stress in rail. The roadbed in the new tunnel referred to has been in service under the heaviest kind of traffic since February, 1909, and has demonstrated fully that if track is made perfectly rigid that it will stand up against the heaviest kind of traffic.

"I am convinced that with a perfectly rigid surface that there will be no pounding and serious damage in railroad track unless some defect existed such as flat wheels. Such conditions, however, are corrected when they do arise, with the railroad rolling stock. I have felt for some time that with the heavy rolling stock that the best ballasted track had practically reached the limit of loading and if the heavy rail is used the supports of same on tie-ballasted track is not like the abutment of a bridge, as

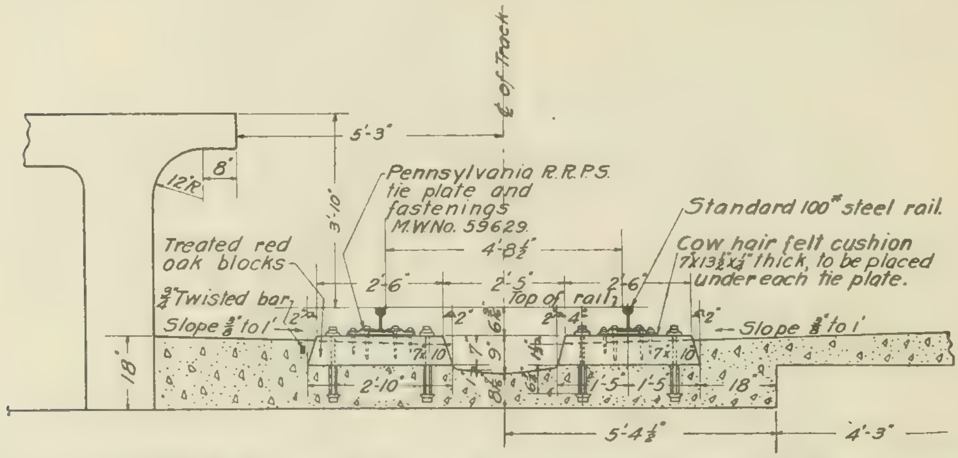


FIG. 7.—CONCRETE RAILROAD TRACK CONSTRUCTION USED IN PENNSYLVANIA TERMINAL, NEW YORK CITY.

ties yield, causing deflections in the rails, and it will be found that even with a heavier rail the stress may be greater in the rail than with a light and less stiff rail section which lends itself more readily to deflection."

The tie blocks are notched at the outer end to form a shoulder and are in place when the concrete is poured. A wedge is driven in between the concrete and the opposite side of the block, preventing the block from being disengaged from the shoulder under traffic. The blocks are replaced by removing the wedge, pushing the block longitudinal of the track so as to disengage the shoulder, and slipping it from under the rail. This work can be done by one man.

Fig. 7 shows the concrete track construction in use in the Pennsylvania R. R. Terminal, New York. A single track length of 14,600 ft. of this type was laid adjacent to platforms. In general the concrete was laid on the rock of the subgrade, but where the subgrade consisted of loose rock backfilling the concrete slab was reinforced. A length of 720 ft. of similar construction is in use in two of the East River tunnels (Nos. 1 and 2),

immediately east of the Long Island shafts. These sections of track are subjected to high-speed traffic.

In 1911, 500 ft. of concrete track support was installed on the Campbell Avenue line of the Chicago Junction Ry. where it crosses the Illinois and Michigan Canal. Details of this construction are shown by Fig. 8. This construction is patented by Louis H. Evans of Chicago and was installed on the Chicago Junction Ry. as an experimental stretch of track, the location being selected with a view to subjecting it to very heavy and continuous traffic. A majority of the stock trains going to the Chicago Stock Yards pass over this concrete track. A thorough examination of the track indicates that although transverse cracks appear in the portion of the concrete above the base of rails, these cracks do not extend down into the supporting slab. In fact, the construction seems to have suffered practi-

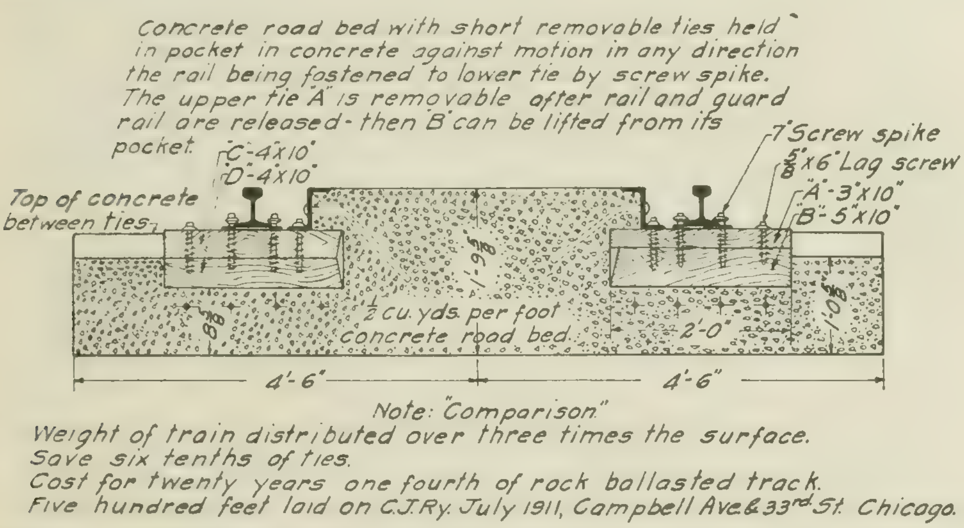


FIG. 8.—CONCRETE RAILROAD TRACKS USED IN THE CHICAGO JUNCTION RAILWAY AT CHICAGO (U. S. PATENT NO. 963,364).

cally not at all from the heavy traffic which it has carried. G. W. Hegel, Chief Engineer, the Chicago Junction Ry., states that there has been no maintenance on this stretch of track excepting to renew a few of the tie blocks. These blocks are spaced 34 in. centers. So great a spacing would not be allowed with the usual ballast and tie type of construction, but with this unyielding track foundation such a spacing has been found to be practicable, since the majority of the deflection in railroad track is due not to the lack of rail stiffness but rather to the lack of any sort of rigid support. Mr. Evans sums up the comparison of this construction with the usual type as follows: Weight of train distributed over three times the surface saves 0.6 of the ties. The cost of twenty years is one-fourth of rock-ballasted track. Fig. 9 and Fig. 10 show the excellent appearance of the track at present.

On the Point Defiance Line of the Northern Pacific Ry., in the State of Washington, there was constructed about five years ago 2000 ft. of track

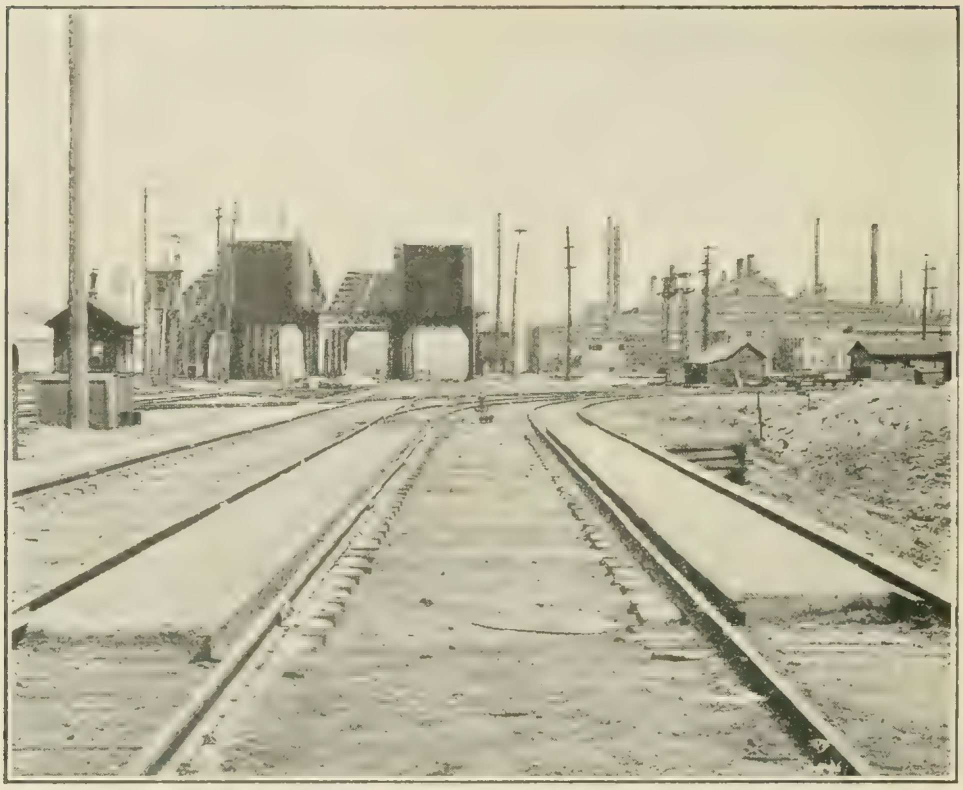


FIG. 9.—CONCRETE RAILWAY TRACK AT THE CHICAGO JUNCTION R. R.

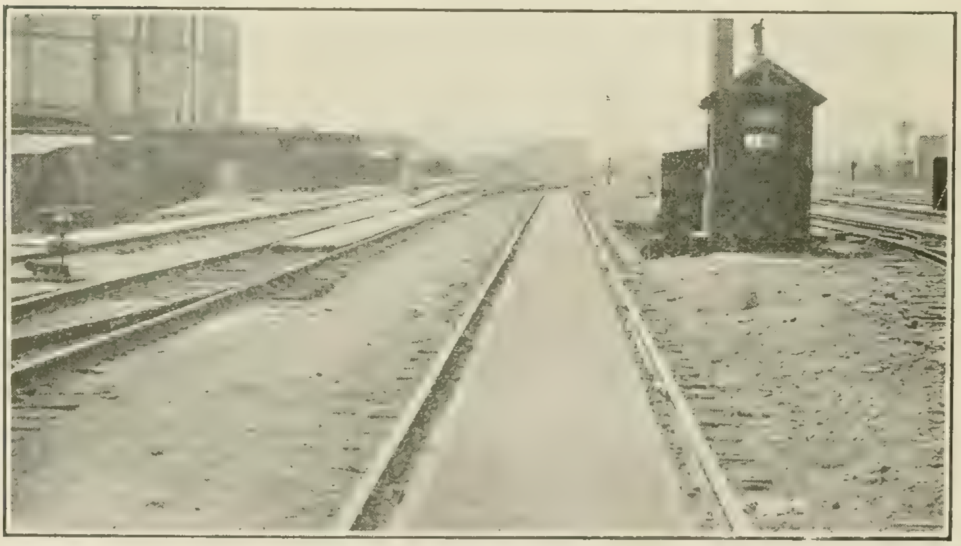


FIG. 10.—ANOTHER VIEW OF THE CONCRETE RAILWAY TRACKS AT THE CHICAGO JUNCTION R. R.

with a concrete slab foundation. Details of the three types used are shown by Figs. 11, 13 and 16. In general, these types consist of a reinforced-concrete slab provided with recesses or troughs for the reception of wood blocks or stringers to carry the rails. Details of Type No. 1 are shown

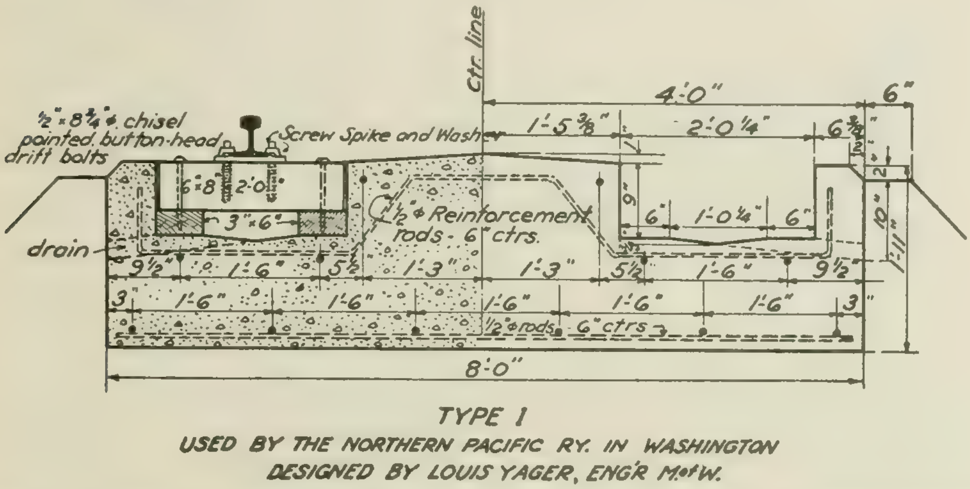


FIG. 11.—TYPE 1, USED BY NORTHERN PACIFIC RAILWAY IN WASHINGTON. (DESIGNED BY LOUIS YAGER, ENGINEER, MAINTENANCE OF WAY.)

FIG. 12.—CONCRETE TRACK CONSTRUCTION, NORTHERN PACIFIC RAILWAY IN WASHINGTON—TYPE 1.

on Fig. 11. The short tie blocks rest on two longitudinal 3 by 6-in. pieces in the bottom of a trough at each side of the slab. The space in between tie blocks is filled with ballast. Drainage of the troughs is provided at intervals. There is a total of 594 ft. of Type 1 construction. The slabs are cast in lengths 16 ft. 5½ in. Fig. 12 is a photograph of this type.

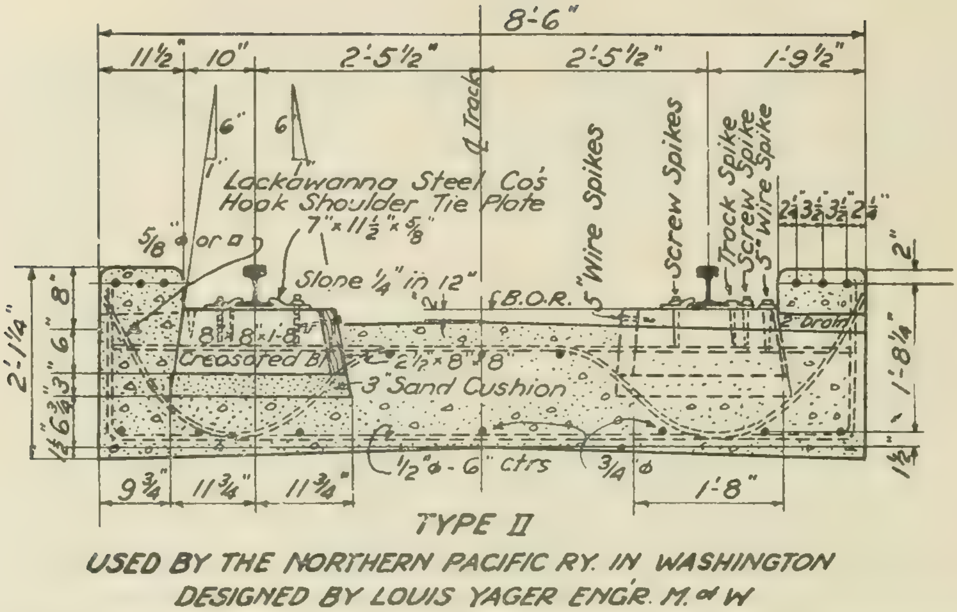


FIG. 13. — TYPE 2, USED BY THE NORTHERN PACIFIC RAILWAY IN WASHINGTON. (DESIGNED BY LOUIS YAGER, ENGINEER, MAINTENANCE OF WAY.)

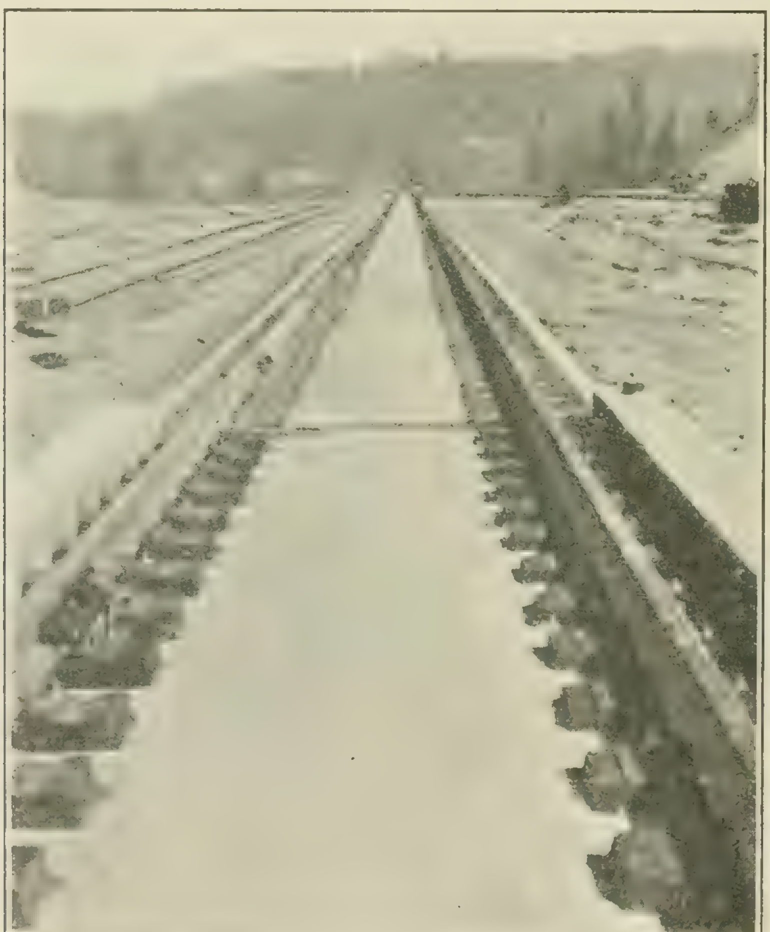


FIG. 14.—VIEW OF TYPE 2 OF THE NORTHERN PACIFIC RAILWAY TRACK.

Type No. 2 (Fig. 13) is 6 in. wider than Type No. 1 or No. 3, and is distinguished by a curb practically equal to height of rail along each side. There is also 594 ft. of single track of this type in service. In this type recesses are cast into the slab into which the tie blocks are placed and

FIG. 15.— ANOTHER VIEW OF TYPE 2 OF THE CONCRETE TRACK CONSTRUCTION OF THE NORTHERN PACIFIC.

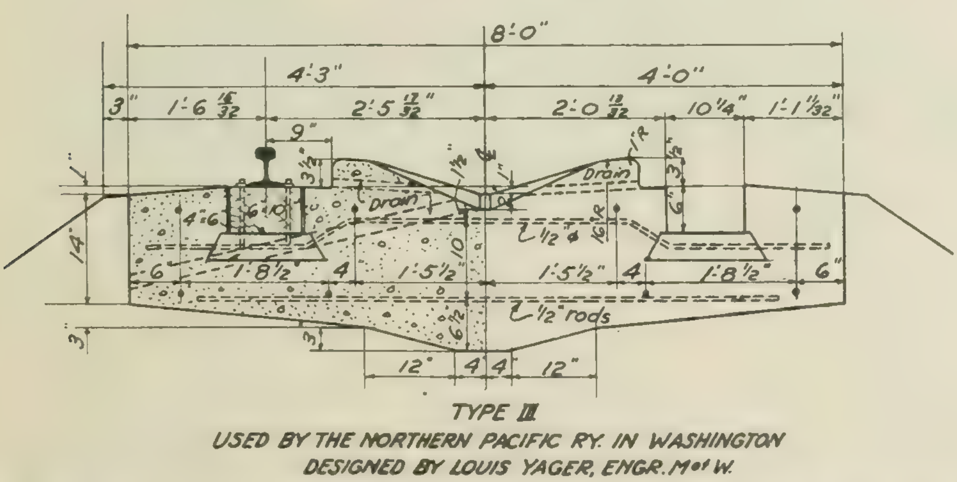


FIG. 16.—TYPE 3 USED BY THE NORTHERN PACIFIC RAILWAY IN WASHINGTON.

wedged into position. The rails are fastened in the usual way with tie plates and screw spikes. In the bottoms of the tie block recesses 3 in. of sand is placed to afford a cushion. On top of this sand cushion the creosoted tie block is placed and wedged at the inner end of the block. L-shaped malleable iron shims are used at the other end of the tie blocks, with one

leg on the L resting on top of and fastened to the tie block. This allows lining and gauging of track. These slabs are cast in widths of 32 ft. 11 in. Figs. 14 and 15 are photographs of this type.

Type No. 3 is shown in detail on Fig. 16. The distinguishing features of this type are that the concrete is carried up $3\frac{1}{2}$ in. near the inside of each rail and sloped to center of track for drainage, and that the rail rests directly on a continuous longitudinal stringer of 6 x 10-in. creosoted fir. These stringers are fastened at intervals by long lag screws driven into

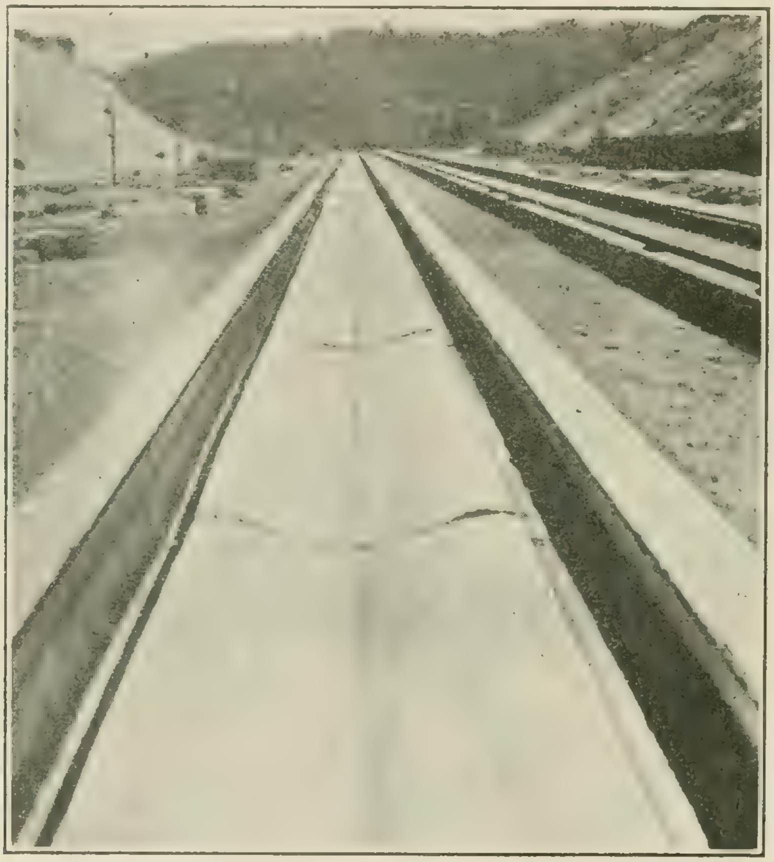


FIG. 17.—A VIEW OF TYPE 3 OF THE NORTHERN PACIFIC RAILWAY TRACK.

wood anchor blocks embedded in the slab. The slabs have a depth of 18 in. at the center and were molded in lengths of 16 ft. $5\frac{1}{2}$ in. There is a total of 810 ft. single track of Type No. 3, and photographs in Figs. 17 and 18 show the appearance of the track in service.

While neither the first cost of the concrete track foundation on the Northern Pacific R. R., nor the annual maintenance cost of it are available, yet the railroad company's officials say that the maintenance has been far below that of track of the ordinary type of construction on the same line. The greater part of this maintenance concerns the wooden blocks to which

the rails are fastened, especially with Type No. 2. In this type the key blocks out in spite of the fact that they are spiked to the tie blocks.

Fig. 19 shows a suggested design for concrete track support prepared by A. D. Whipple and the writer. The form work required for this slab is of the simplest sort. The design of the slab is based on Cooper's E-60 loading with 100 per cent impact. This design does not require any radical departure from the usual type of rail fastenings; that is, a rail clip is used similar to those now standard. There is, however, a very great difference between this rail fastening and the usual cut or screw-spike used with wooden ties. In this concrete design a U-shaped casting is embedded in the concrete and the upstanding legs of the U are tapped to receive the threaded end of holding down bolts which pass through the rail clips, tie

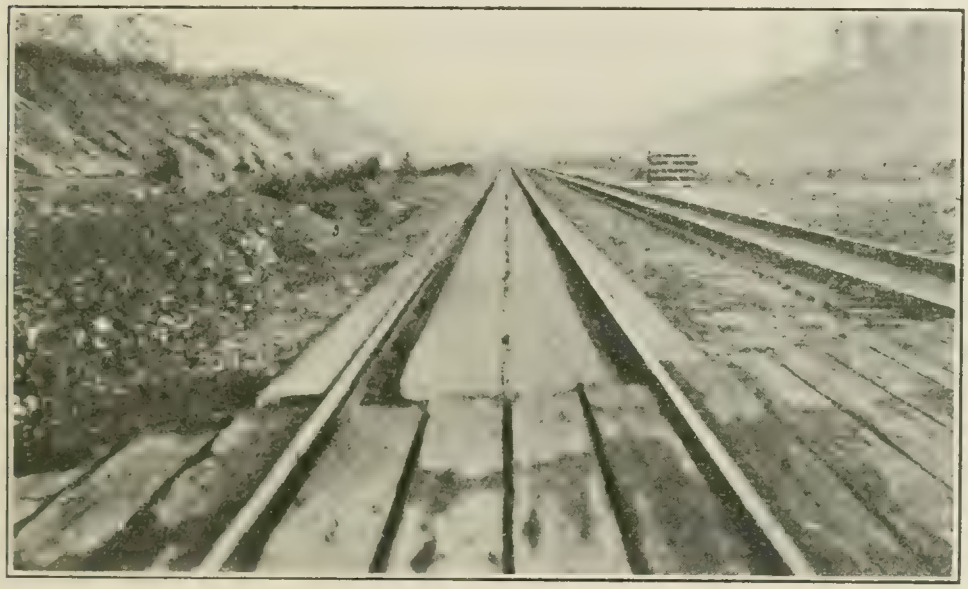


FIG. 18.—ANOTHER VIEW OF TYPE 3 OF THE NORTHERN PACIFIC RAILWAY TRACK.

plate and tie block. A positive fastening, which has great strength against lifting the rail, is thus secured. The use of a block of wood under the rail in this design is at once a concession to the prevalent idea that some elastic medium must always intervene between a track rail and a solid foundation, and a means for easy adjustment of the rail both vertically and horizontally to provide for whatever inequalities may exist in the top of the slab either at the time of placing or subsequent thereto. The latter is the only sufficient reason for using this tie block, and where the slab is placed on well-compacted subgrade the rail may be anchored directly to the concrete, with possibly the intervention of a longitudinal steel plate between the rail and the concrete surface, thus affording the rail a continuous support. In the latter case drainage would be taken care of by finishing the slab to a slight pitch between the rails and furnishing openings beneath the rails at intervals.

ECONOMIC CONSIDERATIONS.

In the final analysis, the use of concrete track construction will be governed by economic factors. Will it pay? Will the economies and advantages derived from it afford an adequate return on the cost of installing it? The answer to this, in so far as I have been able to determine, is very positively yes. Every estimate that has been made by those who have investigated this subject shows even the "visible" income entirely sufficient to warrant the extra expenditure necessary to place such construction under heavy traffic track whether on new construction or on existing lines.

J. W. Schaub estimated in 1899 that the construction shown in Fig. 5 would effect a saving in maintenance and renewals of an amount sufficient

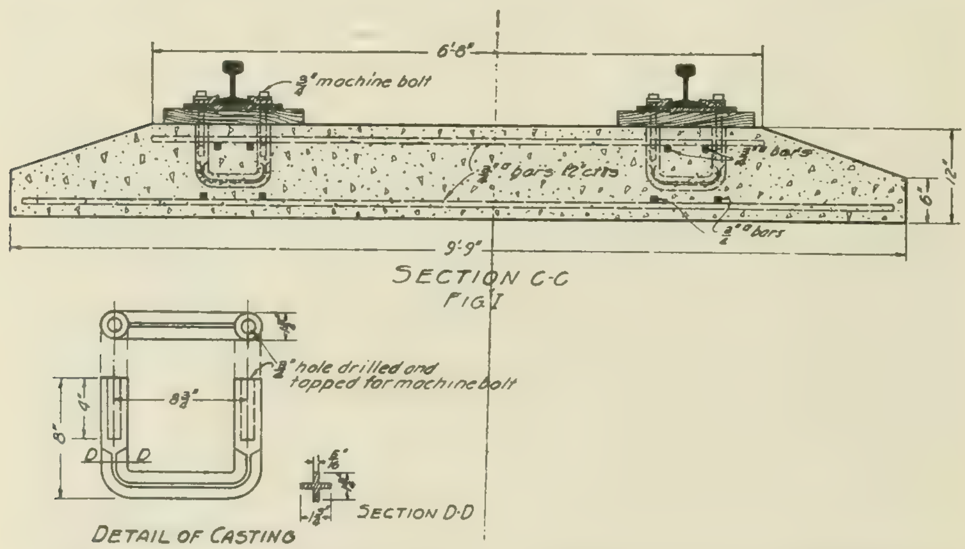


FIG. 19.—SUGGESTED RAILWAY TRACK DESIGN BY A. C. IRWIN AND A. D. WHIPPLE.

to pay 8.8 per cent per year on the added investment. If we credit Mr. Schaub's estimate with the cost of the 12 in. of useless stone under the concrete slab we would have a yearly income of 11.7 per cent on the extra cost. For the design proposed in *Engineering News* staff article mentioned above, Fig. 1, involving longitudinal stringers under each rail it was estimated that the saving on ties alone would pay 3 per cent on the extra cost of the concrete construction, and that therefore the saving in maintenance and renewals of the track other than the ties would be net.

Lincoln Bush, in an article published in the *Railway Age Gazette* of April 23, 1909, shows comparative tables of cost of ballast and tie double-track construction and the concrete slab construction shown on Fig. 6. The cost of the latter was actual cost and the estimated cost of ballast and ties was carefully prepared from data known from experience to be correct. Annual maintenance and renewal costs used were similarly reliable and

the result shows an earning on the extra cost of concrete slab construction of *over 40* per cent per year, due to savings in maintenance and renewals.

A very recent letter from Mr. Bush says: "Bergen Hill tunnel track has now been in service under very heavy traffic over ten years and has fully demonstrated all I believed it would do and is in good condition."

L. V. Morris, Chief Engineer of the Long Island R. R., said in regard to an adaptation of the design shown on Fig. 3 to main line service: "In constructing a new line I cannot conceive that the cost would be over $2.00 per running foot, single track, for a slab 10 ft. wide and 8 in. thick. This would be in the neighborhood of $10,000 per mile and is a very small part of the total cost of a modern roadbed which I have in mind running anywhere from $100,000 to $200,000 per mile, single track. It seems to me that this expense is justified, particularly as it supports practically the only working part of the roadbed. This working part, namely, track, is the most expensive to maintain and upon its condition depends largely wear and tear of the rolling equipment."

Estimates prepared with all possible care on the design presented in Fig. 19, showed an income derived from reduced maintenance and renewals on the concrete type of 18 per cent on the extra cost of constructing new double track, and 12.72 per cent on the cost of placing the slab under existing lines. Neither these figures nor any of those referred to, excepting those of Mr. Schaub, give any credit for saving in motive power due to absence of rail deflection or the reduced maintenance on equipment. In *Engineering News*, Jan. 5, 1899, Mr. Schaub quotes a statement by Dr. H. P. Dudley as follows:

"I am inclined to think that, if the roadbed could be made absolutely unyielding, the springs of the vehicles providing the elasticity, the best results would be had. If the track could be as smooth and relatively as stiff as a planer bed there would be a saving in the cost of maintenance of track and machinery, and in coal consumption. The stiffer the rails, the less the creeping due to the wave which runs ahead of the wheels, the less the wear of the ties due to this motion, the less the destruction of the track and running gear due to the pounding of the wheels and the easier the hauling of the trains.

"Instead of making rail sections simply heavy, I have made them very stiff, which has reduced the deflection, or wave motion, under each of the wheels. Comparing the resistance of the Chicago Limited Express on the stiff 80-lb. rails with that on 65-lb. rails it makes a difference of 75 to 100 h.p."

Mr. Schaub also says that Mr. Dudley designed some rails of 105 lb. per yard, weight nearly 100 per cent stiffer than his 80-lb. rails, and estimated that on fast express trains he would save nearly 200 h.p. as compared with a worn 60-lb. rail.

That there is appreciable resistance to the passage of trains due to track deflection seems beyond question. The wheels are constantly tending to climb to an ever-receding crest. As a matter of fact, while the wheel may progress in a horizontal line, work must be done and lost on the rails, ties, ballast and roadbed which is measured by the force necessary

to produce the deflection and the amount of the deflection. The report of the special committee on Stresses in Railroad Track, above referred to, indicates that for track in "excellent condition" the deflection under applied loads is practically proportional to the load. Deflection curves are also given for locomotive loadings. One of these curves is reproduced herewith (Fig. 20). "By assuming that the work of deflecting the track is lost it may be shown by this curve that the work is equivalent to that required to overcome the resistance offered by a grade of nearly 0.1 of 1 per cent."

In other words, *on level track the deflection allowed by the present ballast and tie type in excellent condition introduces a grade equivalent against the forward movement of the train of approximately one-tenth of one per cent.* For a 3000-ton train this figures out as 1600 h.p. hours lost per 100 miles, which at 1½ cents per h.p. hour for fuel alone *is $24.00 to be charged against track deflection in moving a 3000-ton freight train*

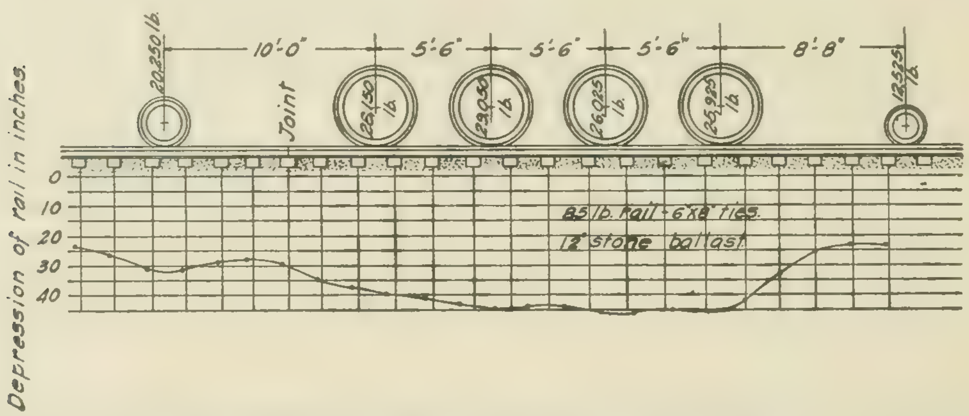


FIG. 20.—TRACK DEPRESSION PROFILE—STATIC-LOAD TESTS ON THE ILLINOIS CENTRAL R. R. WITH MIKADO LOCOMOTIVES.

over an ordinary division of 100 miles. Loss of time, extra labor, etc., will at least double this amount.

How much longer, therefore, will our railroads continue to neglect making a radical change in the present impossible type of track construction—a type that can neither be defended theoretically nor because of its past performance. It is not to be expected that the proposed type of concrete construction would be economical on any but heavy traffic track, but for such track there is a crying need for it. The present general type will doubtless survive on other than the most important lines, but for heavy traffic track there seems to be no other reason why concrete support is not more extensively used than the possible difficulty in financing the first cost. However, such financing should present no unsurmountable obstacle in view of the promising economies and advantages to be derived from the closer approach to ideal track conditions afforded by a strong, permanent non-deflecting concrete track support.

THE DESIGN OF REINFORCED-CONCRETE FUEL-OIL RESERVOIRS.

By H. B. Andrews.*

The scarcity of coal during the late war prompted the United States Government to urge manufacturers and large consumers of fuel to use fuel oil to conserve the coal supply, and to insure the protection of government interests against a shortage of fuel, which would tend to interrupt production of war necessities, through loss of power for manufacturing. The increased use of mineral oil as a fuel, due to this, taught manufacturers that it had several advantages over coal, notably the facility with which it can be handled, the less labor required in its handling, no refuse to be taken care of, the cleanliness of the process and the absence of smoke, and the easy control of the heat. Users of bituminous coal are greatly troubled by spontaneous combustion occurring in large piles, necessitating the expense of frequent re-handling in addition to losses of coal. By the use of oil as a fuel this source of worriment is eliminated.

The comparative cost per b. t. u. between bituminous coal and fuel oil shows an economic advantage for oil as far as the writer is able to obtain testimony.

In order to provide storage for the oil suitable containers had to be installed at each manufacturing plant using it.

In the vicinity of the source of the oil, large reservoirs are provided, some merely excavations in the earth, if it is of an impervious nature, without lining; others, with thin concrete linings and provided with a wooden roof to retard evaporation. Of course, in this method of storage there is considerable loss of oil; that from seepage alone amounting to sometimes four or five per cent, with additional loss from evaporation. The cost of an expensive concrete reservoir would not be warranted on account of the low value of the oil before labor and freight charges have been added to transport it to point of consumption. But when oil has been freighted for hundreds or thousands of miles it acquires an increased value, which necessitates its economical storage and use.

Why Concrete Oil Tanks Are Good.

Until recently steel containers have been generally used for fuel oil. Concrete had not been considered as a suitable material, due to lack of evidence and knowledge of its possibilities. But the practical elimination of steel plates for anything but war purposes forced the use of a substitute for them and reinforced concrete has proven to be satisfactory and superior to steel in many ways for fuel-oil containers if intelligently handled.

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For example, as it is necessary to install most fuel-oil reservoirs under ground, steel tanks rust if not protected. Concrete can be better designed to resist exterior stresses, as hydrostatic or earth pressures. It has the dead weight to better resist upward hydrostatic pressure in soils which often are filled with water. It does not attract lightning like steel, nor, if properly constructed, is it affected by electrolysis. It is a non-conductor of heat and cold, thus retarding evaporation of oil in summer, and also retarding the lowering of the temperature of the oil in winter, an advantage in pumping. In case of a conflagration the oil is much safer in a concrete container than in steel.

But, as previously stated, oil reservoirs of concrete must be designed correctly, the concrete proportioned correctly and mixed and placed correctly in order to get satisfactory results. And by satisfactory results it is meant that there shall be no leakage or seepage when built or thereafter to cause fire hazards or financial loss.

When these necessities have been provided for, reinforced-concrete reservoirs will contain fuel oil of a consistency up to 40 deg. Bé, and practically all fuel oils are below this, the Mexican oils having a specific gravity as low as 16 deg. Bé. For the lighter oils, including kerosene, gasoline or benzine, some provision should be made for a lining of special material.

The design and the location of a fuel-oil reservoir must be considered from various standpoints: that of the owner, who has the location selected for it, and it may be the only available location on his property and not suitable; that of the insurance company who insures the property and whose interest is particularly vital, inasmuch as they have to bear the cost of fire losses should an unwise location cause them; that of the municipality, whose interest is similar to the insurance company in its desire to prevent fires and their spread; that of the company furnishing machinery and pumping equipment, who require that the reservoir shall be installed within practical pumping distance of the furnaces, and finally that of the engineer or contractor, or both, who design and install the reservoir, and who are severally or jointly responsible for its completed condition.

In order to make this paper of maximum value, the writer has requested one of the engineers of the Boston Manufacturers' Fire Insurance Co. to supplement his work by preparing a paper, which is appended hereto, on "The Fire Hazard of Fuel Oil," in which he sets forth in detail the requirements and recommendations of his company and which is in line with the general requirement of insurance companies.

DETAILS OF DESIGN.

The writer has both designed and superintended the construction of many fuel-oil reservoirs of various capacities, and through this experience has obtained knowledge of perhaps some value, and will name some of the requirements in order, and more or less in detail.

(1) *Location.*—The reservoir should be located a safe distance from inflammable structures as far as possible consistent with pumping require-

ments, covered with at least 18 in. of earth, if near buildings, to decrease fire hazards and also to minimize oil evaporation. If distant from buildings it should be at least half underground, and, if possible, the excavated material should be used in banking up around it.

(2) *Size.*—The reservoir should be limited in size for two reasons: First, the necessity of not exceeding a day's working limit in operation of pouring concrete so that joints between operations may be eliminated, and secondly, so that in case of an accident or fire in any reservoir, that too much oil in storage will not be involved. This size limit should not be over 300,000 gals. under most conditions, and the majority of contractors have not the facilities to properly construct a reservoir of this capacity.

(3) *Shape.*—Reservoirs should be circular in shape, the better and more directly to take care of involved stresses and to avert danger of tensile or temperature cracks.

(4) It should be so proportioned and designed as to limit the number of pouring operations of concrete, so as to avoid joints between these operations.

(5) Care should be taken to provide for all exterior stresses, such as hydrostatic pressure from ground water, earth pressure on walls, and roof if reservoir is buried, and also to avoid, as far as possible, concentration of loads on walls or footings; where joints are absolutely necessary, to so protect these joints that there will be no leakage through them.

Regarding hydrostatic pressure, while engineers have found from tests that this pressure in soils is only about 50 per cent of the full head of water, it is not safe to design for stresses less than the full head, as any deflection in the concrete admitting a film of water between the earth and the concrete will produce the full hydrostatic pressure.

(6) So to design the reservoir, piping and vents as to comply with municipal regulations and insurance requirements.

(7) Temporarily or permanently to protect concrete surfaces so that oil will not come in immediate contact with them if concrete is less than six weeks old.

(8) So to design the falsework for holding concrete temporarily in place that it will not fail or be distorted while placing concrete. It is especially necessary to provide for the firm holding of wall forms, as the pressure of several feet of concrete poured quickly as a monolith, is intense, and any give of the forms after the concrete has obtained its initial set breaks up the crystals already formed, allows expansion of the concrete mass, with resultant porosity and loss of strength.

(9) So to design the concrete that it will resist all exterior stresses to which it is subjected and so that it will be oil-proof. And one of the principal features of this design is to make the walls of circular reservoirs in tension, sufficiently thick so that the ultimate strength of the concrete in tension will not be exceeded. It is not meant, of course, to leave out the steel reinforcement so that the stress will theoretically be borne by the

concrete, but, nevertheless, it will be actually borne by it unless some unforeseen weakening of the concrete should throw it upon the steel.

An extended investigation by the writer on high circular concrete standpipes for water, showed that if the concrete in the wall was stressed beyond its elastic limit, or ultimate strength, which is practically identical, vertical hair cracks will appear of sufficient width to admit water into the body of the concrete.

This ultimate tensile strength in a 1: 1½: 3 concrete, from tests made by the writer at the Watertown Arsenal, was 203 lb. per sq. in. Where the concrete is in large sectional areas, and reinforced, this tensile strength will probably be somewhat higher.

If a stress not exceeding 150 lb. per sq. in. is allowed in tension, there will be no danger of these vertical cracks appearing.

(10) So to design the reinforcement that it will take care of all interior and exterior stresses and with fittings to hold it rigidly in place while concrete is being poured. Steel in tension in walls should not be stressed over 10,000 lb. per sq. in. to conform with insurance companies' requirements.

Personally, the writer does not think that it is necessary to figure the stress as low as this, under usual conditions, having satisfactorily constructed many reservoirs, using a stress of 14,000 lb.; but, of course, the lower stress is an additional safeguard against inferior workmanship by inexperienced contractors and from any decrease in bond strength due to oil penetration of concrete. It is probably unwise to depart radically from insurance companies' recommendations.

For other parts of the reservoir the recommendations of the Joint Committee on Concrete and Reinforced Concrete should be followed.

All reinforcing rods in concrete exposed to oil should be of a deformed section for better bonding value.

DETAILS OF CONSTRUCTION.

To carry out these requirements necessitates the employment of competent engineers, experienced in the work, to make the design and specifications and to superintend construction.

The concrete should be no leaner than a mix composed of one part of cement, one and one-half parts of sand and three parts broken stone or gravel. To this mix should be added a "densifier." Hydrated lime has been found economical and satisfactory for this purpose, using ten pounds of dry lime to each bag of cement. The stone must be hard and clean—trap rock, granite or gravel being the best material. The sand must be free from any deleterious matter, and should be well guarded. Cement should be of an established quality.

The concrete should be deposited continuously in concentric layers not over 12 in. deep in any one place.

No break in time of over thirty minutes is permissible in depositing concrete during any one operation, and if any delay occurs, amounting to

thirty minutes or more, the previous surface must be thoroughly chopped up with spades before the next layer of concrete is deposited.

The different operations in pouring are: (1) The pouring of floor and footings; (2) the pouring of entire wall; (3) the pouring of roof.

In small reservoirs the wall forms may be supported so that the footings, floor and wall may be poured in one continuous operation.

An approved joint or dam must be made between the floor and the wall.

When the materials are obtained they should be mixed by a mixing plant of sufficient size and power to carry out each separate prearranged operation without danger of delay during the process.

The materials should be mixed at least two minutes in the mixer, using just enough water to obtain a plastic mix without excess water coming to the surface after concrete is deposited, and a measuring tank should be used so that the amount of water may be kept uniform.

The concrete, when deposited in forms, should be well spaded by at least four competent laborers who are not afraid to use their muscle in compacting the concrete thoroughly and working out the trapped air bubbles.

Reinforcement should be of round deformed bars conforming to "Manufacturers' Standard Specifications for Medium Steel." These bars should be bent or curved true to templates; carefully placed in their predesigned location, and rigidly maintained there by mechanical means.

No laps should be less than 40 diameters and no two laps of adjacent rods should be directly opposite each other.

The forms should be of a good material, strongly made and braced, or held in place by circumferential bands so that no distortion, allowing displacement of concrete during its initial set, is possible.

The surface of the floor should be troweled smooth as soon as it can be properly done. If all previously named precautions are taken, there should be no defects in the wall to correct.

Concrete designed and placed as recommended herein is practically oil-tight, but as oils are somewhat detrimental to fresh concrete, it is advisable to put on an interior wash or coating to protect the fresh concrete from the action of the oil for such a time as may be necessary for it to sufficiently cure and harden.

SODA SILICATE AS OIL PROOFER.

Silicate of soda, while not a permanent coating, has been used satisfactorily for this purpose according to this specification for *oil-proofing*.

The surface of the floor and the interior surface of the wall are to be coated with silicate of soda of a consistency of 40 deg. Bé., as applied as follows:

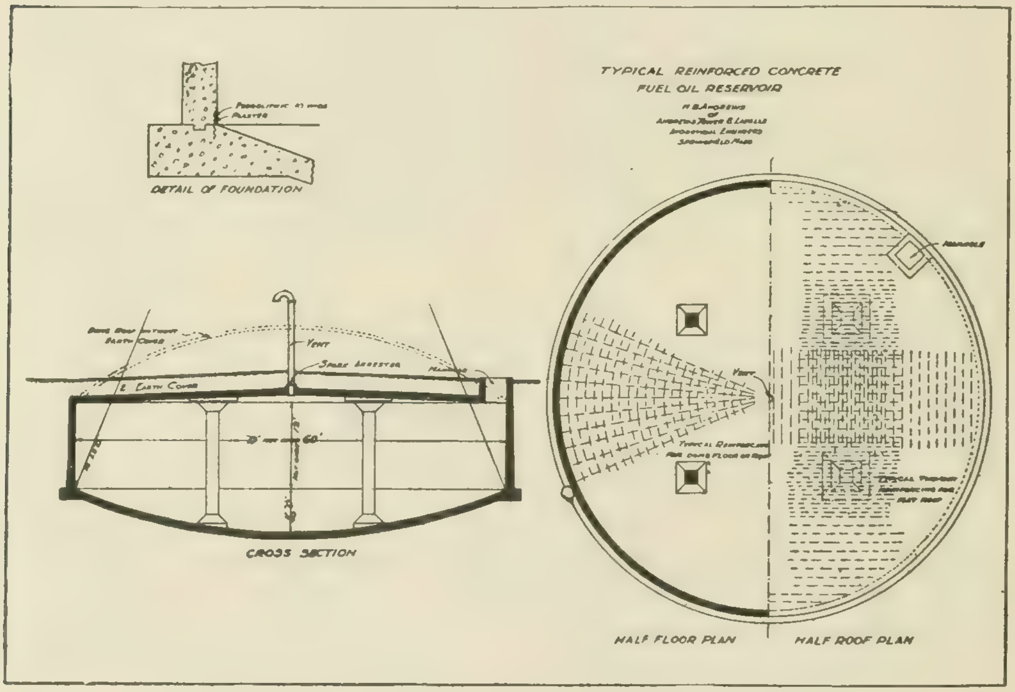
First coat. One part of silicate of soda and three parts water, applied with brush and all excess liquid wiped off with cloth before drying.

Second coat. One part silicate of soda and two parts water, applied as above.

Third coat. One part silicate of soda and one part water, applied with brush and allowed to dry.

Fourth coat. Applied same as third.

The drawing herewith shows a typical concrete reservoir of economical design. The dome roof is economical to construct when an earth covering is not required, and eliminates any concentrated loads which might tend to produce unequal settlement with resultant cracks. The inverted dome at the bottom gives additional storage capacity with only increased cost of excavation, lessens height of wall, requiring less shoring of banks in loose



TYPICAL REINFORCED-CONCRETE FUEL-OIL RESERVOIR.

soils, allows a better drainage of the reservoir than a flat floor, and better resists upward exterior pressure.

The recommended maximum dimensions for this type of reservoir is as follows: Diameter, 60 ft.; height of wall, 12 ft.; rise of roof dome, 1/6 to 1.8 diameter, drop of inverted dome not over 1/10 diameter. The floor and roof should be reinforced both circumferentially and radically to provide against temperature and other stresses.

[Discussion of this paper is included in the group discussion on oil tanks, p. 204.—EDITOR.]

THE FIRE HAZARD OF FUEL OIL.

By J. W. Lord.*

The first use of petroleum as fuel for mechanical purposes was at the refineries of the oil companies, where by-products were used under boilers and for heating. The first application in furnaces at factories insured by the Associated Factory Mutual Fire Insurance Companies was about 1888. The introduction, however, was gradual. As early as 1889, following the well-known policy of the company of advising its members as to safeguards needed for adequately protecting their interests, preliminary rules for installing fuel oil were drawn up by the Boston Manufacturers' Mutual Fire Insurance Co., and in this, as in all similar work, the idea was not to raise arbitrary objections to processes which were deemed advantageous simply because involving a hazard, but rather to advise what experience showed to be necessary in order to remove the hazard or sufficiently guard it so that it would not endanger the establishment itself or the continuity of its operation.

One of the fundamental principles of our organization has uniformly been to assist the manufacturer in every way possible, to the end that consistent development may always be possible in accord with his needs, having due regard at the same time for the requirements of safety.

The early arrangements for supplying fuel oil were crude and unsafe, but experience gained with these, both in connection with their operation and when fires occurred, definitely indicated the course which should be pursued in application of the needed safeguards as originally outlined and which have been perfected more or less in detail since. The early consideration given this matter, combined with the cooperation of the policy-holders, which is always so essential in any mutual work, has resulted in the actual fire loss incurred from the fuel-oil hazard in factories insured by us, being comparatively small and the number of fires few.

Hazards of the Tanks.

As illustrating the hazards and precautions needed a few typical cases may be cited.

Due to failure of oil to pass freely through an oil-supply pipe at the Norwich Bleaching, Dyeing and Printing Co., in 1895, a 2-in. plug had been removed for the purpose of blowing out the pipe, and gases consequently collected in the space under the bleach-house floor, where they were ignited by a lantern in the hands of one of the men. An explosion resulted, followed by fire, with a loss of about $25,000.

As showing the importance of having no overhead oil pipes or any

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unnecessary pipes above ground, the fire at the Washburn & Moen Mfg. Co., in 1896, is illustrative. Here, during repair work, the overhead oil line was being removed, the oil supply having been previously shut off. When disconnection was made, a very small quantity of oil fell upon some ovens located nearby, which, although they had not been operated since the previous afternoon, were still sufficiently hot to vaporize the oil, starting a fire which spread rapidly on the vapor and the somewhat oily surroundings due to the occupancy. As a result the building was destroyed with a loss of $150,000.

The importance of not allowing either gravity or air pressure systems in the supply of oil is illustrated by a fire which occurred at Chicopee Falls in January, 1912, from the discharge of fuel oil into a heated furnace under pressure. The blacksmith attempted to start the furnace after the noon hour and, failing to obtain fuel oil, disconnected the oil pipe at a union. Oil in the tank being under air pressure at once flowed to the burner, and the resulting fire, although small, had serious possibilities.

A few fires where the presence of fuel oil, which has come from other sources on streams, are worthy of note:

In February, 1918, at the Philadelphia Rubber Works, oil reached the property from a refinery several hundred feet above on the Schuylkill River. This oil was set on fire by a locomotive fireman when cleaning his fire on a railroad bridge above, and the tide carried it to the rubber works property, causing a loss of some $4,000.

At the time of the Chelsea conflagration, oil on fire from large storage tanks floated down Chelsea Creek and practically destroyed two railroad bridges and a large sewerage pumping station of nearly fireproof construction, and endangered a large part of the water front of East Boston at that point.

At the Lockport Locks on the Erie Canal, in New York State, waste oil spilled from a large steel works floated down the canal and became ignited from some cause a couple of miles below, and for a time seriously menaced the locks, structures and the surroundings in that city.

A large dock fire was narrowly averted in October, 1918, on the North River, New York, when oil reported to have come from the emptying of fuel oil tanks from army transports, had so thoroughly coated the piling that the under side of the pier caught on fire when the oil was ignited from some cause.

These fires show the need of the installation of oil storage tanks in especially safe manner when located near the bank of a river or other body of water, that leakage may be prevented.

The difficulty of control of fuel-oil fires until all of the oil is consumed is illustrated by one which occurred at Highland Park, Michigan, in 1918. In this case an oil pipe was disconnected without shutting the valve on the supply main, as a result of which the oil was discharged into the heated furnace, where it was at once ignited. Although the oil supply was

quickly shut off outside, the escaping fuel was practically all consumed, making a hot enough fire to melt the glass in the monitors and destroy a portion of the roof.

PRECAUTIONS TO BE OBSERVED.

As an outcome of our investigations and experience, as well as that of the stock companies, it was definitely determined that either a gravity feed for burners or delivering oil under air pressures, which is, in a way, equivalent to gravity system, was not safe and ought not to be used at any plant. Direct pumping to burners is the only safe practice and this has been adopted as a standard. At first, in some cases, the standpipe for main-

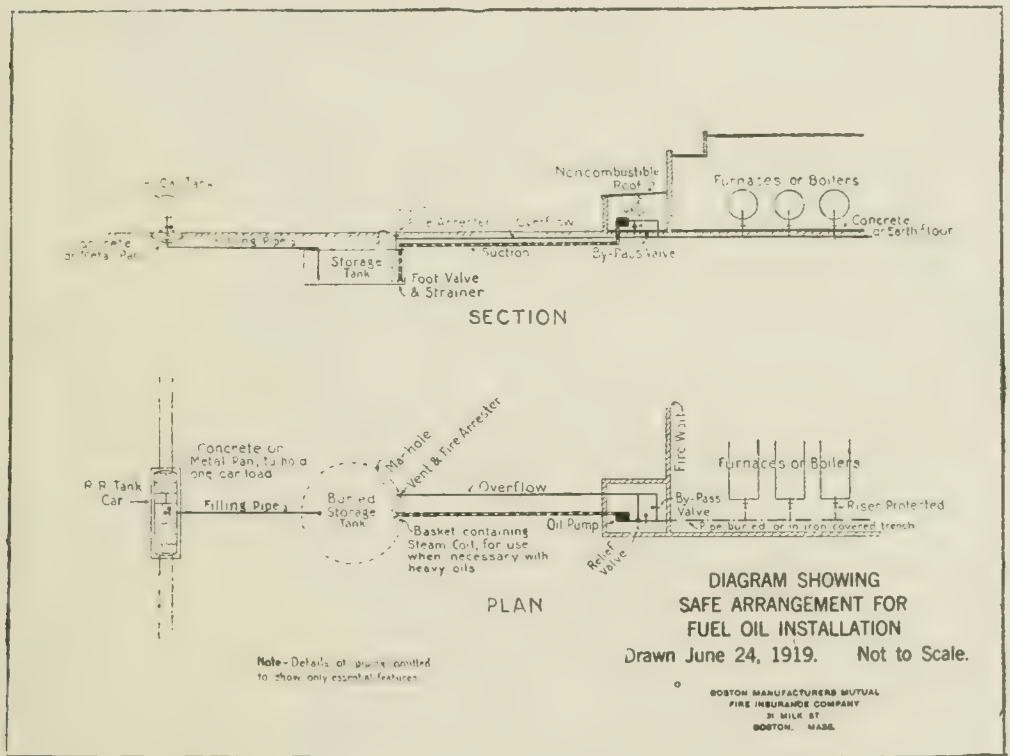


DIAGRAM SHOWING SAFE ARRANGEMENT FOR FUEL OIL INSTALLATION
Drawn June 24, 1919. Not to Scale.

taining a constant head was adopted, but this practice was discontinued, pressure now being maintained by pumping against a relief valve with an overflow back to the tank, by means of which a very uniform pressure is available.

All fuel-oil storage should be underground. The tanks should be located well away from main buildings and not near fire protection mains. Each tank should be independently vented and the vents should be protected by fire arresters. Steel tanks should hold not more than a carload, should be rustproofed and, if set in a pit, should be filled around with sand or sound earth free from acid or ashes.

Quite recently concrete has been successfully adopted for fuel-oil tanks of large capacity and rules for the construction of these have been

formulated by the Inspection Department of the Associated Factory Mutual Fire Insurance Companies. Whether steel or concrete tanks are used all connections should be taken from the top, as otherwise, in the event of settlement, the piping will become broken and the entire contents of the tank discharged, incurring not only loss of the oil but possibility of danger. Great difficulty has also been experienced with concrete tanks in making connections oil-tight when under pressure.

The unloading station for tanks at the railway siding should be located well away from buildings, should be level to avoid possibility of tank car moving after unloading has commenced, and the ground should be sloped so that oil could not reach buildings in event of accident while unloading. The unloading station should have a metal or concrete pan so that oil will not saturate the ground, and large enough to catch the entire contents of a car. The connection for the tank car should either be of pipe with swing joints or metallic hose, thus avoiding the use of rubber hose.

Where storage tanks are installed in batteries there should be valves in connecting pipes between tanks so that any one tank may be cut out at will. All connections into tanks should extend to the bottom so that the end of the pipes will be sealed.

There may be cases where it will be found impracticable to bury the storage tanks, and in a large yard, where the ground slopes away from the buildings, they may be sometimes safely arranged by building the tank above ground and surrounding it with a levee, enclosing a space sufficiently large to contain one and one-half times the entire contents of the tank, with an allowance for an accumulation of snow and water. The levee should be drained and discharge pipes should pass over the levee rather than through it. Under no circumstances should a tank be installed under these conditions near enough to buildings to cause damage if on fire.

Whatever the location of the tank, delivery should never be by gravity. In some cases a small service tank, which should be lower than the pump, may be used, and this may be filled from the storage tank by means of an open connection using a funnel. The service tank is not advised, however, unless necessary, as it complicates the system and introduces the spilling of oil at funnel connections.

The pump maintaining pressure to burners should always be located at higher elevation than the tank and should be so arranged as to prevent syphonic action when the tank is higher than the burners.

FITTINGS AND ACCESSORIES.

All piping should drain back to the storage tank from the burners. The suction pipe should have a foot valve in order to avoid delay in starting, and also to keep the pipe full, and should be so arranged that the foot valve may be readily removed. Piping should be buried and exposed only where rising from the ground to the burners. It should never be overhead. Valves should all be of the outside screw and yoke type so that their position may be seen at a glance.

Fittings should be extra heavy and the entire oil system subjected to 150-lb. hydrostatic pressure before being used, as when a burner or pipe becomes clogged there is a tendency to clear by rapping on it, and if the pipe is weak it will be broken and discharge contents on the floor or into furnaces.

The supply of oil and air or steam for atomizing should be interlocked in such fashion that if for any reason the air or steam for atomizing should fail the flow of oil would also cease. This can be accomplished, where air is used, by driving a rotary pump by sprocket chain from the blower supplying the air for atomizing. A reciprocating air pump should have the air supply taken from the same line as the atomizing supply and beyond the point from which the air for atomizing is taken. Where a steam pump is used and air for atomizing, the steam for the pump should be controlled by a valve actuated by the atomizing air pressure. With a reciprocating steam pump and steam used for atomizing, the steam for both purposes should come from the same source.

The pump house should be made non-combustible with brick or concrete walls and concrete roof. If located inside the main building the pump should be cut off from the remainder of the room by substantial masonry wall. If electrically driven, the motor should be cut off from the pump room by non-combustible partition, and pumps operated by shaft passing through wall, with switches and fuses located in motor room.

There should always be a by-pass around the fuel-oil pump so that all oil may be drained for repairs.

Furnace buildings should, as far as possible, be non-combustible throughout, and with all steel work protected by concrete, and should ordinarily be protected with automatic sprinklers. Where fuel-oil apparatus is located in upper stories of buildings, oil pipe should run up the outside of the building and be protected by encasing in a larger pipe, the furnace being located close to the outside wall so that the run of piping inside may be kept at a minimum. The inside piping should be laid in covered shallow trenches in the floor. Where an oil pipe necessarily runs under a building it should be encased in a larger pipe made up with tight joints pitched to drain outside the building to small wells, so that leaks may be quickly detected.

Where a tank is filled by tank wagon the unloading station should be at the curb so that the tank wagon need not enter the yard, where fire in the tank wagon might cause large loss to buildings in the vicinity and it would be extremely difficult to extinguish it.

Although a considerable amount of the fuel oil now being used is of heavy grade, it should be understood that under certain conditions the oil presents all the hazards of a lighter oil. In order to use the heavier grades at all it is necessary to heat them, which results in fluidity and introduction of the hazards of the lighter oils. The apparatus as installed will be permanent and it is entirely possible that, with the discovery of new oil fields, lighter grades may be used. There have also been cases where unsuitable

oil has been received and attempts made to remedy the situation by the addition of benzine. The quantity of benzine added would necessarily be indefinite and the hazard of the oil thus materially increased. It is, therefore, important that with any oil system installed safeguards be adopted to care for definite hazards.

On the principle that "an ounce of prevention is worth a pound of cure," the efforts of the Boston Manufacturers' Mutual Fire Insurance Co. have been directed largely towards prevention and preventative methods rather than towards methods of extinguishing oil fires, although these also have been thoroughly investigated. None of the means of extinguishing ordinary fires are entirely satisfactory when applied to oil fires and, in fact, there does not at the present time appear to be any certain method. The most satisfactory apparatus, however, is the foam extinguisher, in which two solutions are mixed and pumped onto the surface of the oil, producing a smothering effect and in cases complete extinguishment.

An important feature of fuel-oil storage is the necessity of taking precautions to avoid loss of life when cleaning out the tanks. The vapors are extremely poisonous and, as they are heavier than air, it is only possible to remove them from the tanks by pumping out. With the presence of sludge, however, which continuously gives off the gas, this method will not suffice. The only safe way then is for workmen to wear either a diver's helmet or a gas mask and this precaution should always be taken.

Where the steam in a plant is entirely maintained by the use of fuel oil, any material interruption to this service would result in shutting down the steam plant, and consequently the heating facilities would be suspended. In order to be assured of reliability in oil supply the pump set should be in duplicate.

RULES IN USING FUEL OIL.

Summing up the whole question of the use of fuel oil, the general rules which should be followed are few and simple and are briefly stated in the following:

1. Storage tanks should be underground and well away from main buildings and fire mains, and so located that oil cannot flow from them onto the surface of streams in the vicinity.

2. Unloading stations for either tank cars or tank wagons should be safely located and constructed.

3. All piping should be buried and graded to drain back to the tank.

4. Supply to burners should be by pumping, and oil and air or steam for atomizing should be interlocked.

5. The entire system should be mechanically strong to prevent possibility of breakage.

6. Scrupulous care should be observed in maintaining neat and orderly conditions around the tanks, pumps and oil-burning apparatus.

7. Plans for proposed fuel-oil installations should always be submitted

for approval to the underwriters having jurisdiction before commencing work.

It may be fairly stated that while the use of fuel oil presents a very definite fire hazard, it is a controllable one as is attested by the excellent fire record in the factories insured by the Boston Manufacturers' Mutual Fire Insurance Company.

[Discussion of this paper is included in the group discussion on oil tanks on p. 204.—EDITOR.]

TESTS OF CONCRETE TANKS FOR OIL STORAGE.

By J. C. Pearson and G. A. Smith.[1]

During the past year a series of experiments have been conducted at the Bureau of Standards for the purpose of determining the limitations of concrete for oil storage. These experiments have consisted in the main of permeability tests on small concrete tanks containing various types of oils, but consideration has also been given to the possible injurious effects of the oils on the concrete, and to the use of coatings which might be effective in preventing or reducing the penetration of oils into and through the body of the concrete.

The tests herein described belong for the most part to three series: the first on tanks of 1:2:4 concrete nearly filled with oil but under no pressure; the second on tanks of $1:\frac{2}{3}:1\frac{1}{3}$ concrete, similar to the concrete used in the construction of concrete ships, and filled with oil as in the first series; and the third on tanks of $1:\frac{2}{3}:1\frac{1}{3}$ concrete under a 25-ft. head of oil. The test tanks were of the same shape and size throughout, and the construction was similar in all cases except in certain details providing for tight covers in the pressure tests. The tanks were molded bottom upward in steel forms, the details of which are shown in Fig. 1. The form consists essentially of a nearly cylindrical core shaped like an inverted bucket with a slight flare from the bottom to the top, and an outside cylindrical shell. The dimensions of the finished tanks are $20\frac{1}{2}$ in. high over all, and 23 in. outside diameter. The inside dimensions are $14\frac{1}{2}$ in. diameter at the bottom, $15\frac{1}{2}$ in. at the top, and $16\frac{1}{2}$ in. deep. The tanks therefore have an average wall thickness of 4 in. and a capacity of approximately $13\frac{1}{2}$ gal. when filled to the brim. No reinforcement was used in the construction of the tanks. In the tanks that were subjected to pressure, $\frac{3}{4}$ x 5-in. bolts were cast in the top rim to hold the sealing plate.

The concrete of the first series of tanks was composed of 1 part cement, 2 parts Potomac river sand, 0 to $\frac{1}{4}$ in., and 4 parts Potomac river gravel, $\frac{1}{4}$ in. to $1\frac{1}{2}$ in., mixed 5 minutes in an open pot type of mixer. 8 per cent of water by weight of total dry materials was required for a good workable consistency. The concrete was puddled into the forms by hand.

In the second and third series the concrete was composed of 1 part cement, 2.3 part Potomac river sand, 0 to $\frac{1}{4}$ in., and $1\frac{1}{3}$ parts Potomac river gravel, $\frac{1}{4}$ in. to $\frac{1}{2}$ in., mixed as above described. 12 per cent of water was required for a good workable consistency. After placing the concrete in the forms, the latter were vibrated by quick light hammer blows for about 15 minutes.

As a rule the forms were removed on the day after pouring and the tanks were immediately filled with water and allowed to stand for two weeks. The tanks were then emptied and allowed to dry for at least two weeks before being put under test.

[1] United States Bureau of Standards, Washington, D. C.

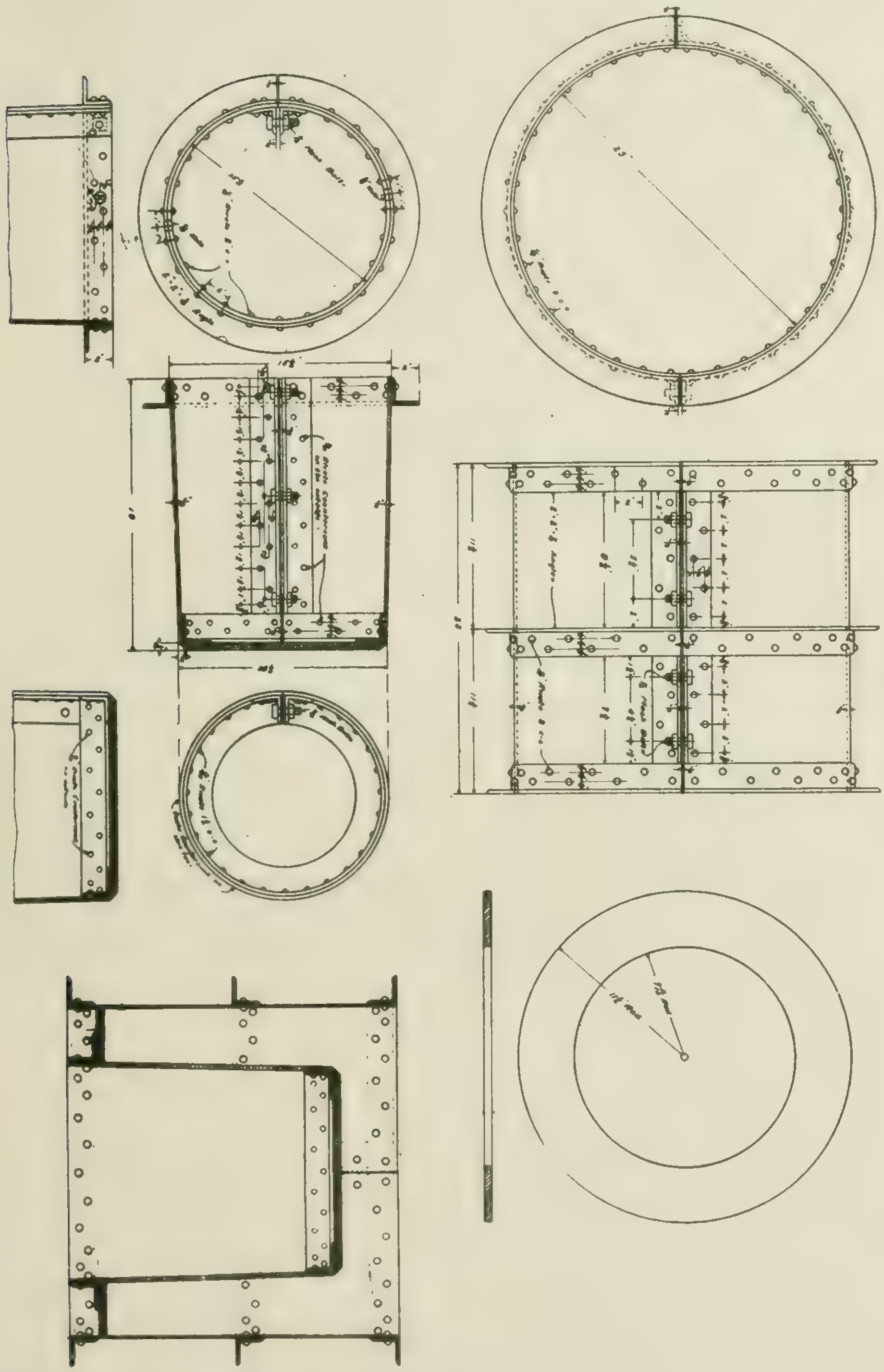


FIG. 1—STEEL FORMS USED IN CONCRETE TEST TANKS IN OIL PENETRATION EXPERIMENTS.

THE FIRST SERIES OF TESTS.

In the first series of tests, started in May, 1918, eighteen varieties of oil were placed in as many tanks and stored in an open shed. Eleven of these were mineral oils with gravities ranging from 0.742 to 0.959, that is, from commercial motor gasoline to very viscous residuum oil, five were vegetable oils of different types, and two were animal oils. (See Table I for complete list.) In each of these tanks were placed three 6 x 12-in. cylinders of 1:2:4 concrete, dried and weighed, in order to determine the absolute absorption, depth of penetration, and crushing strength after long storage in oil. The tanks were covered with glass plates pressed down upon a ring of neat cement paste spread upon the top rim of the tanks. These plates became loose as

TABLE I.—CONCRETE OIL-STORAGE INVESTIGATION. SERIES 1.

Tank No.	Kind of Oil.	Specific gravity of oil at 60° F.	Specific gravity Baumé, degrees.	Viscosity at 20° C.	Volume of oil absorbed, cu. in. at 60° F.		Average depth oil penetration, ins.		Ultimate compressive strength, lb. sq. in.	
				C. G. S. units.	Days 200	385	Days 200	385	Days 200	385
1	Mineral	.781	49.3	0.011	17.6	18.0	0.9	1.1	5567	4039
2	"	.934	19.9	2.064	14.0	15.5	0.2	0.4	4581	3820
3	"	.801	44.8	0.014	17.5	18.4	0.5	1.1	3989	1981[1]
4	"	.882	28.7	0.142	17.7	15.9	0.2	0.4	3047	3866
5	"	.908	24.4	0.427	12.4	13.4	0.4	0.8	4478	3537
6	"	.742	58.7	0.004	20.6	16.4	0.8	0.9	4189	4594
7	"	.959	16.0	13.926	11.1	12.7	0.2	0.2	4105	4497
8	Cottonseed	.926	21.2	0.667	8.1	7.2	0.3	0.2	4012	3554
9	Mineral	.827	39.3	0.033	16.2	15.6	0.5	0.9	4608	3997
10	"	.854	33.9	0.063	17.5	18.7	0.4	0.6	3855	3082
11	"	.852	34.3	0.053	14.7	18.1	0.4	0.6	3537	3427
12	Paraffin	.894	26.6	0.401	16.7	15.8	1.1	1.5	3335	3554
13	Linseed, Raw	.912	23.5	0.318	10.4	11.8	0.5	0.8	3855	3121
14	Linseed, Boiled	.921	22.0	0.447	10.3	4.4	0.4	0.2	3216	3572
15	Neatsfoot	.917	22.7	0.954	7.3	8.8	0.1	0.2	4250	3409
16	Lard	.917	22.7	1.216	9.3	9.0	0.2	0.2	3371	4279
17	Turpentine	.870	30.9	0.019	16.1	18.8	0.6	1.0	3370	3972
18	Cocoanut[2]	.898	26.0	....	1.0	1.5	0.1	0.2	4109	4259

[1] Omitted from mean, see text page.
[2] Physical properties of this oil determined at 55° C.

the cement hardened, but nevertheless made fairly close fitting covers for the tanks.

It was planned originally to make quantitative measurements of the rate of loss of the various oils as stored in the first series of tanks by means of a simple gaging device shown in Fig. 2. A square pine stick was wedged into the top of the tanks, and the increasing distance of the oil surface from this stick would indicate roughly the rate of oil loss. It was found, however, that in order to have any value commensurate with the effort of gaging the oil losses far greater care and refinement would have to be employed in the measurements, account would have to be taken of the coefficients of expansion of the oils, and precautions would have to be taken to avoid losses by evaporation of the lighter oils. The storage of the concrete cylinders in these tanks with the intention of removing them for test at different periods also

introduced another obstacle to the quantitative determinations, and it was therefore decided to continue the tests for an indefinite time, and observe qualitatively the effects of the oil on the concrete, and obtain what information was possible from the tests of the cylinders.

One of the three cylinders in each tank was removed after 200 days immersion, the surface oil wiped off, and the increase in weight determined. This increase in weight divided by the specific gravity of the oil gave the total volume of oil absorbed by the cylinder; the compressive strength of the latter

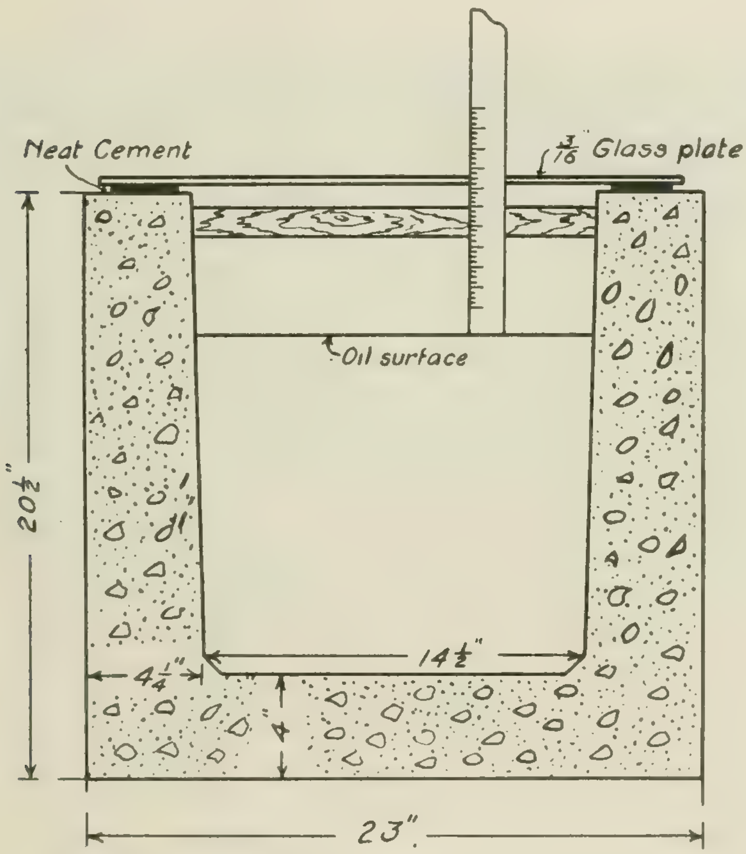


FIG. 2—GAGING DEVICE USED IN FIRST SET OF TESTS.

was then determined and the average depth of oil penetration observed as closely as possible.

The second cylinder in each tank was tested at the age of 385 days as above described. All the data of these tests are given in Table I. Lacking sufficient of the original oils to supply the gradual losses and to make up for the volume of the cylinders removed, clean, dry coarse gravel was recently introduced to bring the oil surface up to approximately its original height in the tanks, and the tests will be continued as simple exposure tests as long as conditions permit.

RESULTS OF TESTS OF FIRST SERIES.

The most important result of this series of tests is that after a year's storage of the various oils in concrete tanks, a careful examination has failed to reveal any disintegration of the concrete except in two cases. These exceptions are cocoanut oil and lard oil, the former having attacked the surface of the concrete to such an extent that the finer sand grains can now be quite readily scraped off with the finger nail. The lard oil has merely roughened the surface of the concrete to a slight extent. The compression tests of the cylinders stored in these oils for 385 days show no reduction in strength. The penetration is also slight, and the disintegration appears to

FIG. 3—TANKS IN SERIES 1.

be confined entirely to the surface of the concrete immediately in contact with the oil.

The crushing strengths of the individual cylinders show considerable variation, due to the fact that the ends are not always true and parallel, but the following averages indicate no deterioration from storage in the various oils.

AVERAGE COMPRESSIVE STRENGTHS (LB. PER SQ. IN.) OF 6 x 12-IN. CYLINDERS.
Cured 2 Weeks in Water and 2 Weeks in Air.

	Time of Storage. 200 Days.	385 Days.
Mean of 3 cylinders stored in air	3660	4130
" " 11 " immersed in mineral oils	4120	3840
" " 2 " " " animal oils	3810	3840
" " 5 " " " vegetable oils	3710	3700

Saponification effects were noted in four of the tanks of this series, particularly in those containing linseed oil. The deposits on the surfaces of these tanks were quite heavy, but the concrete was hard and smooth, and showed no deterioration. There was a slight deposit on the surface of the tank containing neatsfoot oil, and a somewhat heavier deposit on the tank containing lard oil.

An interesting feature in connection with these qualitative tests is the slow creeping effect of the medium weight oils over the surface of the concrete. This is plainly shown in Fig. 3, where the first and third tanks from the right

FIG. 4—A PORTION OF THE TEST TANKS IN SERIES 1 AND 2.

show the oil stain which has gradually crept up over the top of the tank and down the outside. Penetration spots may also be observed on these two tanks. Since the picture was taken (about a month after the tests were started) these tanks, which contain kerosene of 0.78 and 0.80 specific gravity respectively, have become completely saturated. Some of these tanks may be seen in the background of Fig. 4.

THE SECOND SERIES OF TESTS.

The second series of tests, started in October, 1918, was a part of the investigative work for the Concrete Ship Section of the United States Shipping

Board involving a rich concrete similar to that used in the construction of the ships. Twenty-one tanks were used in this series, sixteen containing mineral oils ranging in specific gravity from 0.649 to 0.970, five containing vegetable oils, and one containing water. (See Table II for complete list.) These were arranged for quantitative determinations of the losses by penetration, and involved a complete knowledge of the coefficients of expansion and other characteristics of the various oils. The apparatus for gaging the height of the oil surface is shown in Fig. 5. Special attention may be called to the method of sealing the glass covers to reduce evaporation loss to a minimum, to the provision for temperature measurements, and to the gaging device.

TABLE II.—CONCRETE OIL-STORAGE INVESTIGATION. SERIES 2.

Tank No.	Kind of Oil.	Specific gravity of oil at 60° F.	Specific gravity Baumé, degrees.	Viscosity absolute at 20° C.	Loss in 40 Days.	
					Cu. in.	Cu. in. sq. ft. of exposed concrete.
30	[1]Residuum	0.970	14.3	2.347	6.7	1.2
31	[1]Fuel	0.936	19.6	1.790	15.2	2.6
32	"	0.911	23.7	0.236	22.9	4.0
33	"	0.877	29.6	0.052	31.0	5.4
34	"	0.848	35.1	0.033	42.5	7.4
35	"	0.824	39.9	0.021	60.0	10.4
36	Kerosene	0.803	44.4	0.015	37.3	6.5
37	"	0.784	48.6	0.011	52.6	9.1
38	Gasoline	0.743	58.4	0.003	71.4	12.4
39	Kerosene	0.810	42.8	0.015	32.5	5.6
40	Gasoline	0.729	64.7	0.005	81.2	14.1
41	"	0.692	72.3	0.004	122.1	21.1
42	"	0.649	85.7	0.003	160.0	27.7
43	Mid. Cont., Crude	0.859	33.0	0.101	34.0	5.9
44	Light Mex., Crude	0.936	19.6	0.816	18.8	3.3
45	Paraffin	0.881	28.9	0.266	22.4	3.9
46	Cottonseed	0.922	21.8	0.702	44.8	7.8
47	Linseed, Boiled	0.938	19.3	0.644	7.3	1.3
48	Linseed, Raw	0.934	19.9	0.474	13.0	2.2
49	Turpentine	0.869	31.1	0.011	59.3	10.3
50	Peanut	0.917	22.7	0.854	....	....
51	Water	1.000	10.0	0.010	47.2	8.2

[1] The viscosities given for these oils are 55 degrees centigrade.

Without going into the details of the computation of the oil losses, it was found necessary to reduce the oil volumes at each gaging to the corresponding volumes at a base temperature of 60° F. and to compare these volumes with the original volumes at 60° F., in order to derive the accumulated losses at each observation. In this manner each loss determination was independent of the others, and accumulative error was avoided. The magnitude of the errors involved have been carefully considered, and it may be shown that the error in gaging the height of the surface is the chief source of the irregularity in successive observations.

RESULTS OF THE SECOND SERIES OF TESTS.

The losses of the mineral oils stored in tanks Nos. 30 to 38, inclusive (except for tank No. 35), are shown by the curves in Fig. 6. These losses

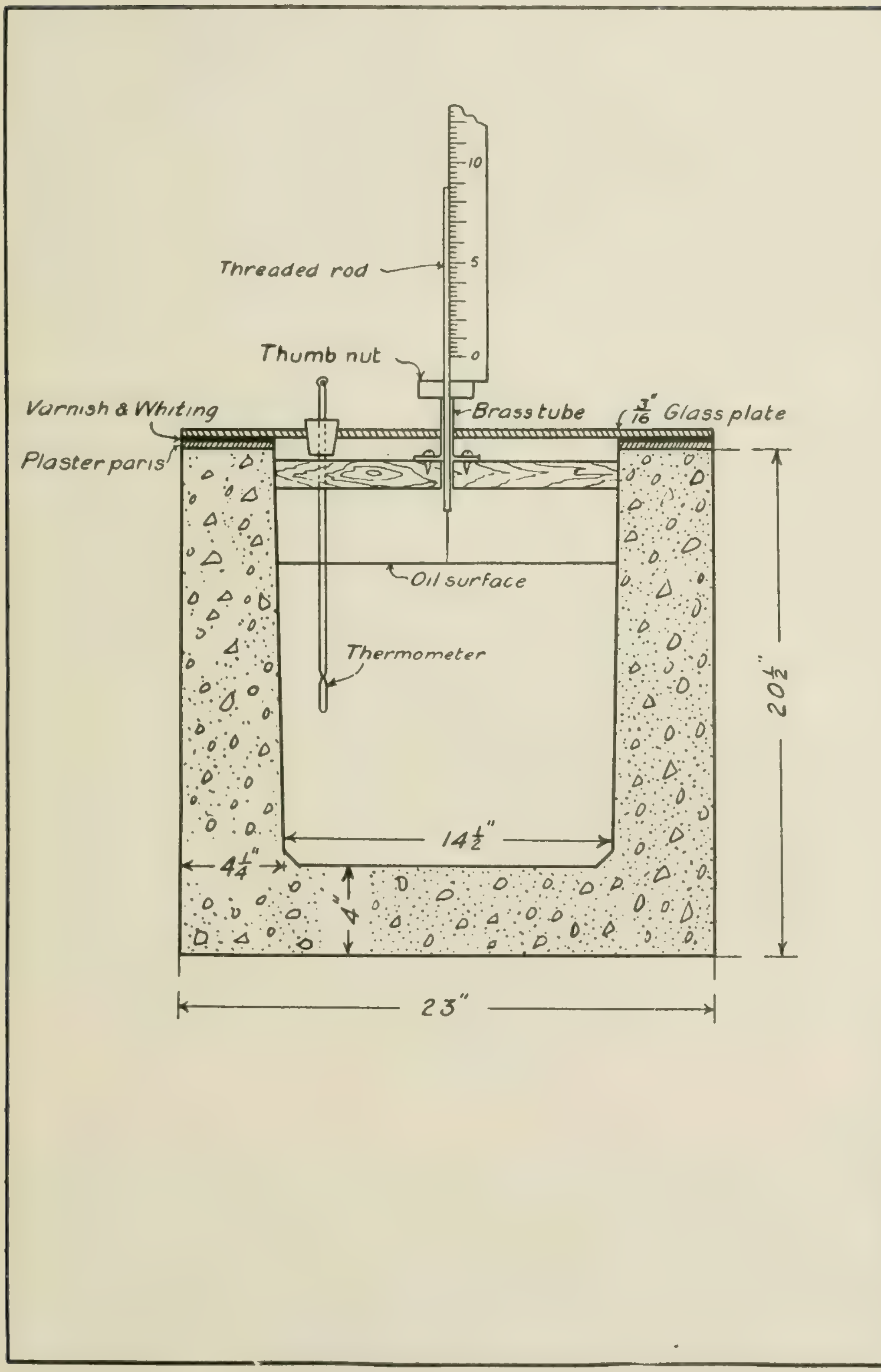


FIG. 5—APPARATUS FOR GAGING THE HEIGHT OF OIL SURFACE IN SECOND SET OF TESTS.

are expressed in cubic inches of oil per square foot of tank surface and are plotted for a period of approximately 5 months. The ordinate of any point on a given curve gives the accumulated loss, and the slope of the curve gives the rate of loss. The data on the curves give the specific gravities of the oils referred to water at 60° F., the specific gravities in degrees Baumé, and the viscosities in C. G. S. units at 20° C., except as noted.

If the tanks were absolutely alike in porosity we should expect the oil losses to increase as the viscosities diminish, and this is seen to be the case

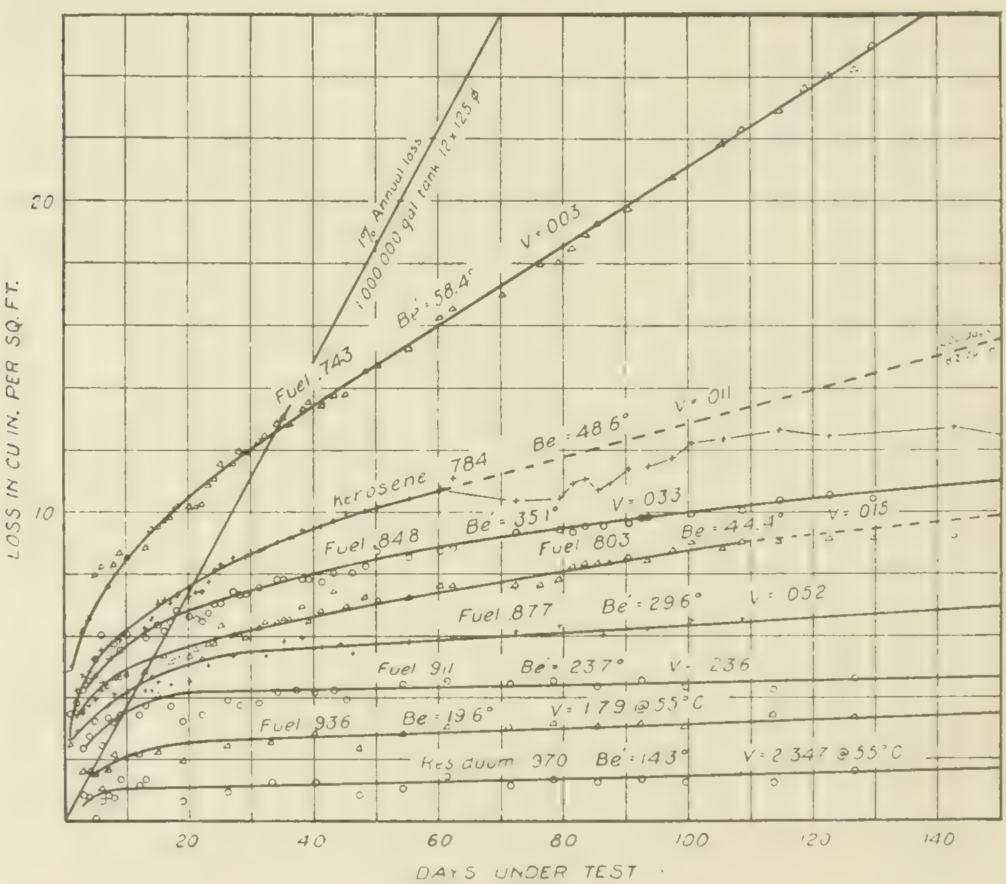


FIG. 6—OIL LOSS CURVES IN SERIES 2.

with one exception, viz.: the positions of the curves for oils with viscosities of 0.015 and 0.033 seem to be reversed. This is undoubtedly due to difference in the quality of the tanks. Since the specific gravities of the oils, expressed in degrees Baumé, usually increase as the viscosities diminish, the general relation may be more familiarly expressed by saying that as a rule the oil losses and rates of loss are higher, the higher the Baumé gravities. The drop in the curve of the 49° kerosene was found to be due to water getting into the tank from a leak in the roof of the storage shed. This water was removed and replaced with kerosene, and a correction made on the assumption that the loss during the period that water was in the tank was

unaffected by the presence of the water. This correction was made at a date beyond the limit of the diagram, and indicates that the actual loss curve should have followed approximately the dotted line.

For the purposes of comparison a straight line has been drawn on the diagram, which represents an annual loss of 1 per cent of oil from a 1,000,000 gal. concrete tank 125 ft. in diameter and 12 ft. deep. Expressing this loss as cubic inches per square foot of tank surface (i. e., of bottom and wall), and

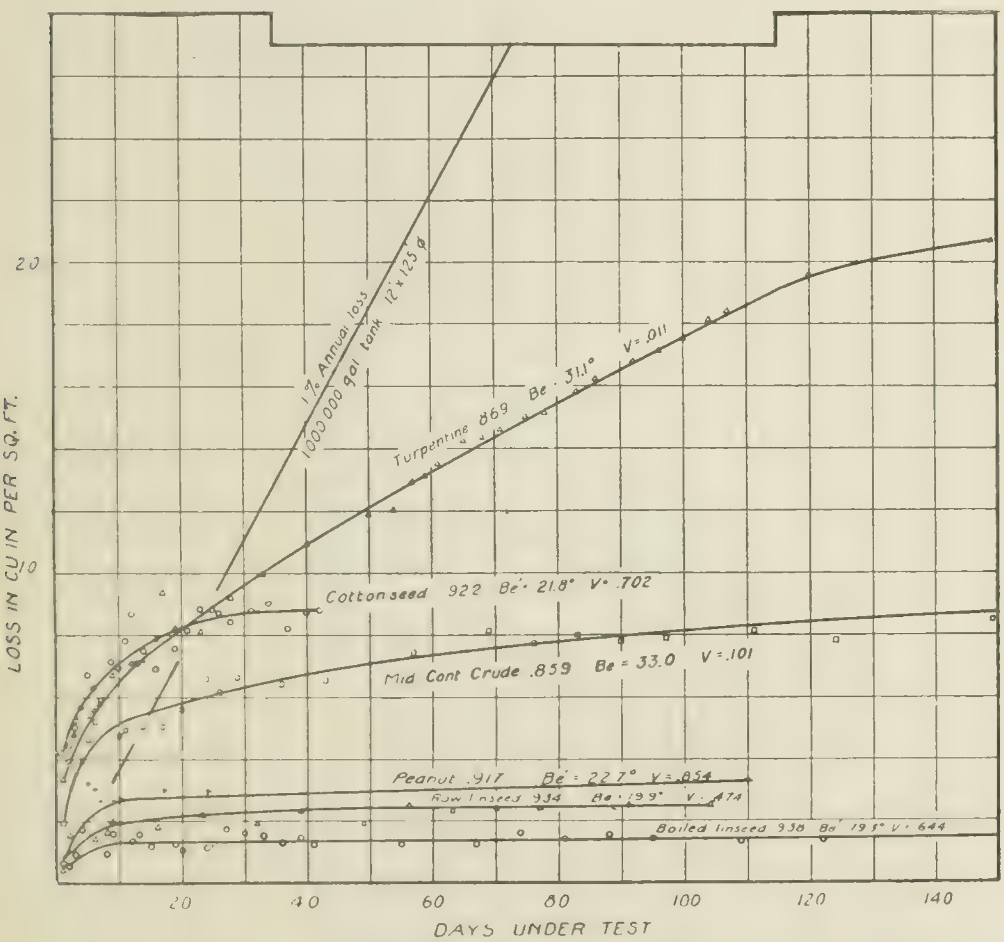


FIG. 7 OIL LOSS CURVES IN SERIES 2.

assuming a constant rate of loss, this straight line would be the loss curve for the large tank. The slope of this line is decidedly greater than that of any of the curves in Fig. 6, which indicates that in order to obtain a loss of 1 per cent per year from a tank of this size and dimensions the rate of loss must be decidedly greater than the rate of loss in these tests. It may be noted that the upper curve in this diagram is that of a light mineral oil corresponding very nearly to the gasoline obtainable at automobile service stations.

In Fig. 7 are shown the loss curves for the vegetable oils and one mineral oil. All of these show relatively low losses except the turpentine, which

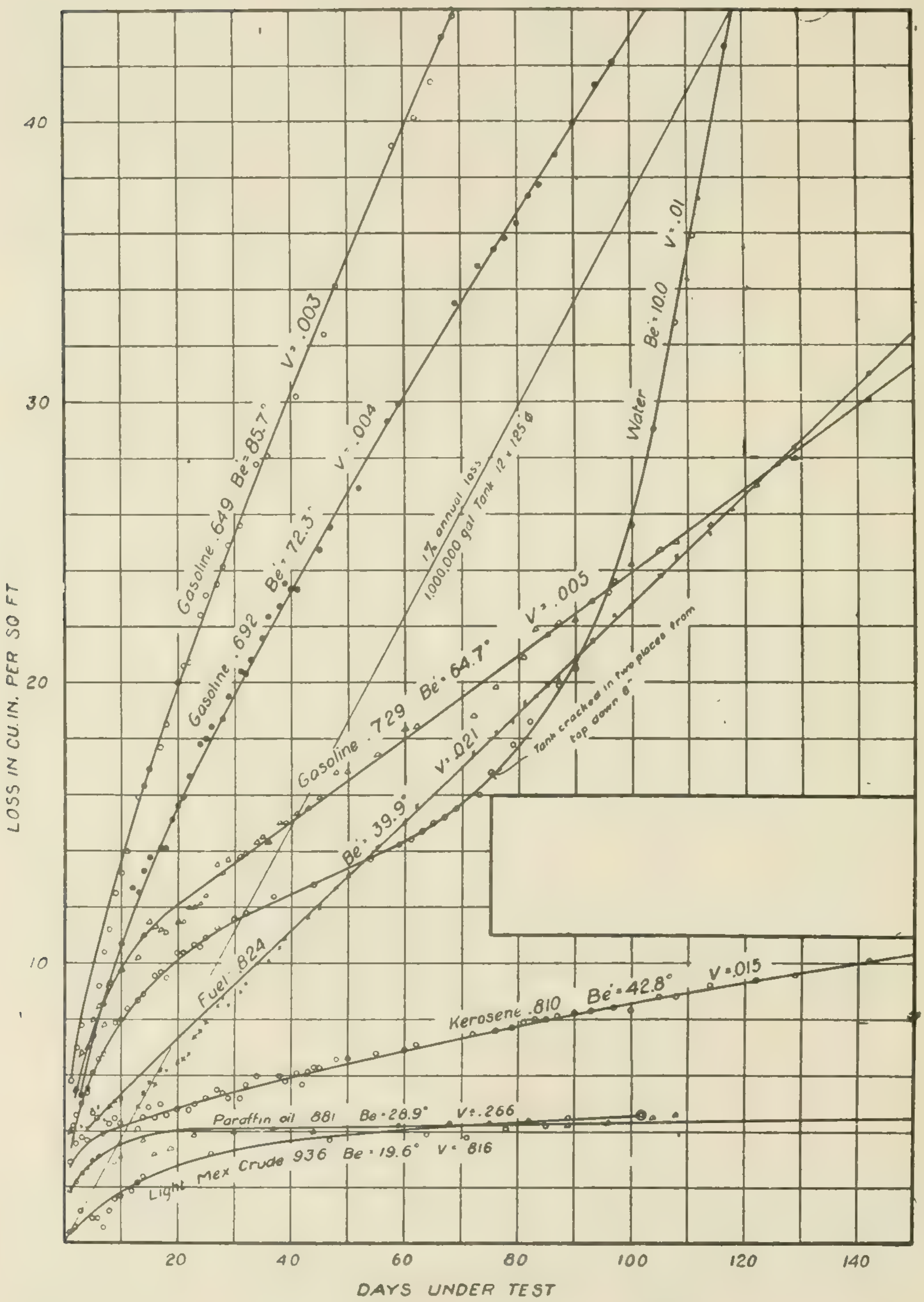


FIG. 8—OIL LOSS CURVES IN SERIES 2.

appears to have a somewhat higher loss than mineral oil of the same viscosity (See Fig. 6). The cottonseed oil solidified about the forty-second day of the test, due to cold weather, and no further readings were possible during the period covered in the diagram.

Fig. 8 contains the loss curves of the remaining oils of this series, three gasolines, kerosene, light fuel oil, heavy crude oil, paraffin oil, and water. It will be noted that the gasoline losses are relatively high and in the order of

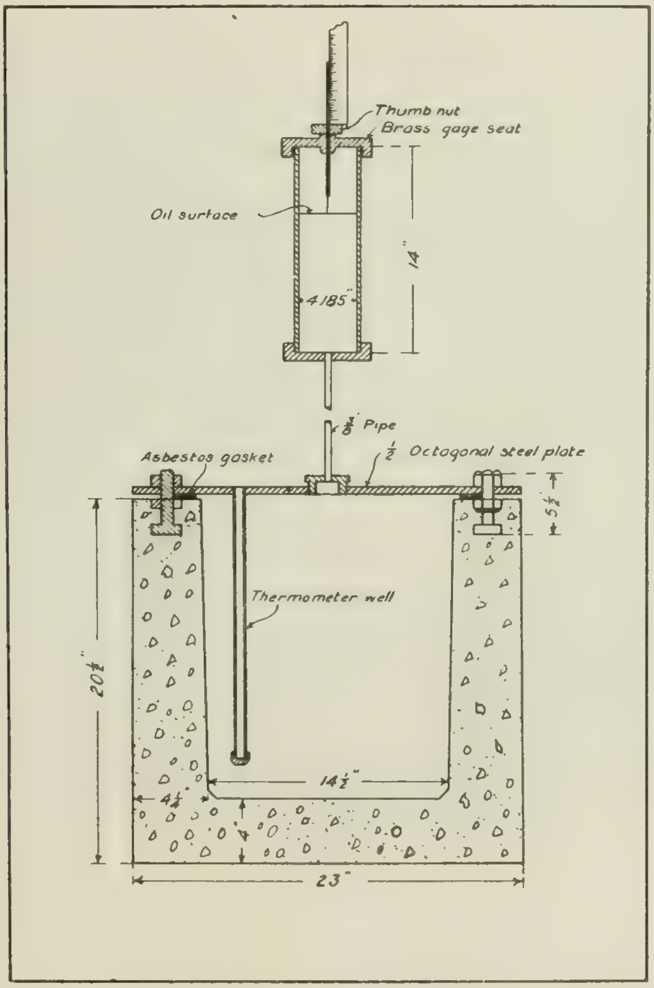


FIG. 9—GAGING METHOD USED IN THIRD SERIES.

their Baumé gravities; the curves for the oils with viscosities of 0.015 and 0.021 are reversed in position, the former being lower, and the latter higher than they might be anticipated to be in normal concrete tanks.

The water tank in this series showed an interesting development. The test was made during the early part of the winter, the tank being kept indoors and not far from a radiator. After about $2\frac{1}{2}$ months two fine vertical cracks were discovered extending from the top of the tank down about 8 inches.

The only explanation which suggests itself is that sufficient compressive stress had developed in the wet concrete of the inside of the tank to produce tension failure in the dry outer layer. The cracks could not be detected on the inside of the tank.

THE THIRD SERIES OF TESTS.

The third series of tests, also started in October, 1918, was made upon tanks similar to those of the second series, that is, of $1:\frac{2}{3}:1-\frac{1}{3}$ concrete, but

FIG. 10—TANKS 60, 61 AND 66 OF SERIES 3, CONTAINING, RESPECTIVELY, WATER, 72° GASOLINE AND 86° GASOLINE.

with the oils under a pressure head of 25 ft. The purpose of these tests was to determine the quantitative losses of oils under the maximum pressure that would be likely to be encountered in commercial storage, either in tanks or ships.

Fig. 9 shows the general set-up of the tanks in this series. These tanks were cast with $\frac{3}{4}$ x $5\frac{1}{2}$ in. bolts in the top to hold the $\frac{1}{2}$-in. steel cover plate. A $\frac{3}{8}$-in. pipe was carried from the center of the plate to a height of 25 ft., ending in a gaging cup 4.185 in. diameter x 14 in. deep. These cups were machined on the inside and on the upper end, and provided a very accurate and satisfactory means of gaging the oil surface. The gaging error involved

in this series is only $\frac{1}{16}$ the magnitude of that in the second series, and the loss curves are decidedly more free from irregularities on this account. In this series the chief errors arise from the uncertainties in the temperature determinations, and later two additional temperature wells were installed at different distances from the tank wall.

In this series it was most important that all leaks in the system be eliminated, and the difficulties naturally centered around the joint between the

FIG. 11 TANKS OF SERIES 3, CONTAINING FROM LEFT TO RIGHT 49° KEROSENE, 40° FUEL OIL, 30° FUEL OIL AND 14° ROAD OIL.

cover plate and the tank. This joint was successfully sealed by the use of an annular gasket of asbestos, rubber, and metal, treated with spar varnish.

RESULTS OF THE THIRD SERIES OF TESTS.

In this series seven tanks were originally put under test, one containing water and the remainder mineral oils ranging from very light gasoline to very heavy road oil. These tests were continued for $1\frac{1}{2}$ months. In Fig. 10 are shown tanks Nos. 60, 61, and 66, containing water, 72° gasoline and 86° gasoline, respectively. In Fig. 11 are shown tanks 62 to 65, containing 49° kerosene, 40° fuel oil, 30° fuel oil and 14° road oil, respectively. In this series the loss of water and of the two heaviest oils was remarkably low, that of the

gasolines and kerosenes was high and irregular. Practically complete penetration of the kerosene tanks is shown in Fig. 11, the photograph being taken when the test had been under way for a period of 40 days. On account of these apparently inconsistent results it was decided to change the oils about and to start a new series of tests. Additional tanks were also constructed and repeat tests were made on these same oils in the new tanks.

The data of tests of the third series which have been run a minimum of 40 days are given in Table III. In connection with these data attention may be called to three or four outstanding points:

First, large variations in the losses for a given oil in different tanks are noted in several instances.

TABLE III.—CONCRETE OIL-STORAGE INVESTIGATION. SERIES 3.

Tank No.	Kind of Oil.	Specific gravity at 60° F.	Specific gravity Baumé, degrees.	Viscosity at 20° C. C. G. S. units.	Oil loss in 40 Days, cu. ins. per sq. ft. of tank surface. Individual.	Average.
60	Water	1.00	10.0	0.010	5.8	7.45
67	"	....	....	....	9.1	....
65	Residuum	0.970	14.3	2.347[1]	1.1	1.1
64	Fuel	0.877	29.6	0.052	4.1	9.6
71	"	....	....	....	15.1	....
62	"	0.824	39.9	0.021	32.5	15.6
69	"	....	....	....	12.5	....
70	"	....	....	....	14.6	....
[3]66	"	....	....	....	3.0	....
[3]60[2]	[2]Kerosene	0.810	42.8	0.015	2.0	2.0
[3]61	"	0.784	48.6	0.011	9.5	25.1
63	"	....	....	....	49.2	....
72	"	....	....	....	16.5	....
61	Gasoline	0.692	72.3	0.004	29.8	53.6
[3]62	"	....	....	....	43.9	....
68	"	....	....	....	87.2	....
63	"	0.649	85.7	0.003	21.0	31.1
66	"	....	....	....	41.3	....

[1] Viscosity of this oil is at 55° C.
[2] This tank is coated with spar varnish.
[3] Tanks put under test second time with different oil.

Second, the average losses for different oils are not always consistent; e. g., the 72° gasoline shows higher penetration than the very light 86° gasoline.

Third, the tanks themselves sometimes show peculiar variations when retested with different oils. E. G. Tank No. 63 was first tested with 49° kerosene, showing a very high loss; when retested with 85° gasoline it showed a very low loss. Similarly tank No. 66 was first tested with 86° gasoline, showing a high loss; when retested with 40° fuel oil it showed a remarkably low loss. In general all of the tanks put under test for the second time show relatively lower losses than in the original test. This may be due in part to saturation of the concrete during the earlier test.

The mean loss curves for the various oils are plotted in Fig. 12. In this diagram the straight line representing the assumed annual loss of 1 per cent from a 1,000,000-gal. tank is interesting as a possible dividing line between loss curves which would be considered excessive and those which would be

permissible. In adopting such a criterion consideration would have to be given to many points, and it is hardly worth while at this stage of the investigation to enter into a discussion of the matter. The point which the authors

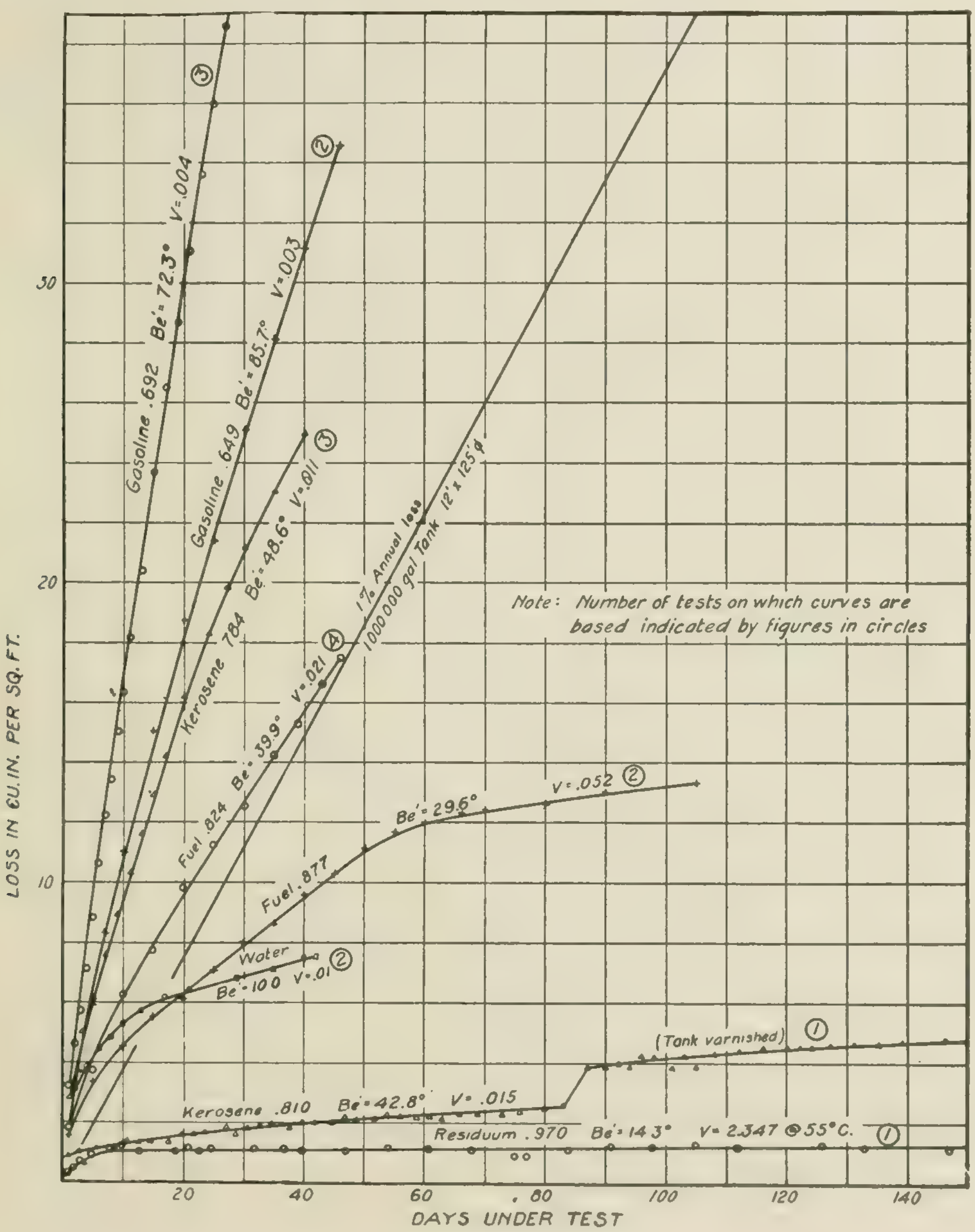


FIG. 12—OIL LOSS CURVES IN SERIES 3.

desire to make is that assuming a permissible rate of loss of oil, and knowing the capacity and dimensions of any given tank, a rate curve can be drawn expressing the loss in units of volume per unit of tank surface. When more test data are available on the retaining power of concrete for various oils

under various conditions, curves of this type should be helpful in estimating the efficiency of well built concrete oil tanks.

One other indication from the quantitative tests of Series 2 and 3 may profitably be considered in later tests. In nearly every case the loss curve of any given oil in any given tank has become practically a straight line within two months after the beginning of the test. This means that the rate of oil loss in an untreated tank becomes constant within a comparatively short time, and in general there is nothing to be gained by continuing the test further. This statement would not apply, however, to tests of coatings, the durability of which can only be determined by long exposure tests.

OILPROOF COATINGS.

The Bureau has not as yet taken up the testing of oilproof coatings to any considerable extent. Preliminary experiments conducted for the Concrete Ship Section of the United States Shipping Board indicated that spar varnish was the most promising material for a coating exposed to both oil and water, and one very encouraging result with this material is shown in Fig. 12. Tank No. 60, after being under a water pressure test for 46 days, was dismantled, dried, and given two coats of 8 per cent solution of magnesium fluosilicate and two coats of spar varnish. It was then filled with kerosene of 43° Be. and has shown a remarkably low loss under a pressure head of 25 ft. The test has now been continued approximately six months and is still in progress. We have no knowledge of the present condition of the coating, but it is apparently effective.

One or two other tests have been made with spar varnish which are less promising than the foregoing, but there has been some question as to conditions under which the coatings were put under test, and the results may therefore be passed until further data are available.

While the subject of oilproof coatings is exceedingly interesting and important, we have felt that the retaining value of concrete tanks for oils must depend fundamentally upon the concrete itself, and that probably no coating short of an all metal lining will be found capable of converting a poor tank into a good one. Whatever results may eventually be obtained with oilproof coatings, we believe that the only logical method of testing such coatings is by comparison with the untreated concrete, and that a general knowledge of the limitations of the latter is necessary before a program of coating tests can be intelligently prepared.

SUMMARY.

The most important results of the investigation to date may be briefly summarized as follows:

1. Various mineral oils covering practically the entire range of fuel oils have been stored in concrete tanks approximately 13 months, apparently without injuring the concrete in the slightest degree.

2. 6 x 12 in. concrete test cylinders have been stored in these oils during

the same period and have shown no appreciable diminution in compressive strength.

3. A number of vegetable and animal oils have been stored successfully in concrete tanks for a period of 13 months, and only two, cocoanut oil and lard oil, have appreciably attacked the concrete.

4. Quantitative losses of fuel oils have been determined in $1:\frac{2}{3}:1\frac{1}{3}$ concrete tanks under pressures of 12 to 15 in. of the oils, and in a smaller number of tanks, under a pressure head of 25 ft. These measurements indicate that even under the latter conditions heavy and medium weight fuel oils can be stored in concrete of this character without excessive losses. The storage of kerosenes and gasolines under these conditions will probably prove uneconomical, unless some impervious coating can be found which will be durable under long exposure to the lighter oils.

5. In a single test of six months duration, spar varnish has apparently been effective in successfully retaining a 43° kerosene of 0.015 viscosity. The loss during this period has been practically negligible.

DISCUSSION OF PAPERS ON CONCRETE OIL TANKS.

President W. K. Hatt in the chair.

Mr. Eldridge. MR. H. W. ELDRIDGE.—I believe that Mr. Andrews has allowed too high a percentage of hydrate of lime for the mixture used. The concrete mix is 1: 1½: 3, and he recommends 10 lb. of hydrated lime to the bag of cement. I believe that this amount of hydrated lime would tend to cause hair cracks, would increase the shrinkage cracks and would have a tendency to allow the oil to get under the outside coating and attack the concrete, or if not attack it, at least scale off the outside coating of mortar. In the last year or so I have the opportunity to examine a good many concrete oil storage tanks, and in a number of instances where these tanks have not been entirely satisfactory, have traced the cause to the use of too much hydrated lime. In these tanks all the other methods used seem to be standard in every respect; this was really the only cause that would make the tanks unsatisfactory.

Mr. Wason. MR. L. C. WASON.—The summary of Mr. Pearson mentioned all too briefly the effect of lard oil and one other oil on concrete. I would like to ask if he will describe more in detail the appearance of the surface of the concrete and state if it was seriously affected by any of the oils, and and also if any of the oils were affected by the concrete?

Mr. Pearson. MR. J. C. PEARSON.—The most serious attack was by cocoanut oil. I think that was first found out a month after the oil was put in. The oil was solid, or partly solid, about half the time, anyway, so it was expected there would be no penetration and no effect, but it was quite serious. In about a year we could scrape the finest sand off the surface of the finish; nothing more serious than that. The attack by the lard oil was much less noticeable at the end of the year. I would say it could be described by saying that concrete surface was roughened a little bit; it was not possible to scrape off any sand and you would hardly more than notice that the surface was roughened. As far as the effect of the concrete on the oil was concerned, there were some that showed a saponification effect. That was the case particularly with linseed oil. In those cases, however, there seemed to be no effect on the concrete. There was a sort of coating or deposit possibly around the surface, some little oxidized oil, which might have been a few hundredths of an inch thick. Lard oil is the only oil that seemed to act both ways; that is, there was a slight saponification and also the effect upon the tank was appreciable. I would say, however, that as far as linseed oils are concerned—the oils that would pass as good oil—there seemed to be no serious effect. There was a little effect that could be detected from the original examination of the oil, but they were still satisfactory.

Mr. Vandyke. MR. G. W. VANDYKE.—I would like to ask Mr. Pearson whether he does not think that the apparent loss of oil would be greatly decreased if the interior of the tank was given a coating of one part of cement to one of sand, and after the coating had dried, it was given a treatment of two or three coats of magnesium silicate.?

Mr. Pearson. MR. J. C. PEARSON.—I think it is very possible; I am not in position

to answer the question directly, however. These were very rich concretes. I doubt whether one coating would be any more impervious than that concrete. You have to bear in mind a number of things, the quality of the concrete, no structural defects, and you might say perfect mixing, because it was placed in a small unit; so far as the coating proposition is concerned, we would like to know the effect on the untreated tanks. First, we have not had any too much faith in coating, having been rather sceptical about starting in, but we shall do that and I would not be surprised if sodium silicate would help us considerably. **Mr. Pearson.**

MR. S. C. HOLLISTER.—I want to refer to the first paper by Mr. Andrews, first of all, in connection with the stress test which he recommends. He suggested a working stress of 150 lb. per sq. in. in concrete and about 10,000 lb. per sq. in. in steel; 10,000 lb. in steel is the common stress for water and oil tanks that have been built so far. If that occurs in the tanks as a working stress, of course the stress in the concrete would be sufficient to have caused a tensile failure in the shell. It does not seem reasonable to me that the cracks occurring in the concrete can be avoided, because of the fact that with a rich mix, there is a certain shrinkage, and that any amount of shrinkage that may exist will cut down the ultimate tensile strength of the concrete before cracks have been formed. That is to say, if you adopt 150 lb. per sq. in. on tension in the concrete and have a considerable amount of shrinkage cracks, it will fail before you reach that 150 lb. per sq. in. tension. Furthermore, stresses in a tank of this sort involve careful reinforcement in the vertical direction, and, unless that joint is made carefully, great difficulty will be found at the connection of the side walls with the base, both from bending moment from the top and from shear due to side pressure. Mr. Andrews suggested a construction joint being formed in the time, not to exceed thirty minutes, from the time of placing the layer immediately below. A construction joint can be made with a much longer wait than this and can be made satisfactory. There is only one thing that must be looked after, and that is the amount of laitance which is formed, which can be governed by cutting down the amount of water. The practice which has been used in ship construction will probably apply in the case of tanks better than any other case. In ordinary practice it would seem that the experience gained in ship construction should be followed as nearly as possible in order to obtain a water-tight or oil-tight wall. **Mr. Hollister.**

MR. C. H. UPHAM.—A few moments ago it was suggested that 10 per cent of hydrated lime was an excess. I quite agree that probably this 10 per cent is an excess, but after studying this subject for two or three years, I have never noted any case where this increases the air cracks. I will say that that subject is now under observation by several committees of the A. S. T. M. I am a member of one of them and we expect to go into this matter quite thoroughly. Referring to the number of cracks that do occur in these concrete tanks, I might say that we have found the method of curing to have a direct relation to the number of cracks that occur in the concrete. I know of one particular instance where, during very hot weather, we secured a certain number of cracks in a concrete road and, under the **Mr. Upham.**

Mr. Upham. exact same conditions, we changed the method of curing and we cut the number of cracks down to possibly 10 per cent of what were occurring under the first method of curing.

Mr. Andrews. MR. H. B. ANDREWS.—I want to mention in order some of the questions that have been brought up in regard to hydrated lime. I superintended the construction of nine reservoirs, each using 10 per cent of lime. Parts were exposed above ground and there was no evidence of any shrinkage whatever or any cracks occurring inside or outside of those reservoirs. The lime is more plastic and easier to work around the reinforcement, and also the concrete sets better. In regard to the contraction joint, I did not intend to set any particular time; I meant that thirty minutes should cover the continuous operations of pouring concrete around the walls; that is, not over thirty minutes should elapse before the concrete is poured up against the previously placed concrete of the floor. As to the steel in tension, I have always used 14,000 lb.; 10,000 is the regulation of the insurance companies, but I do not feel that if the concrete is designed in tension in the wall with a maximum of 150 lb. stress, there will ever be any vertical or shrinkage cracks occur. It never has been my practice; we have plastered the inside of reservoirs so that any minute cracks would show, and those have always developed when the concrete was spread up to around 250 or 300 lb.

Mr. Chapman. MR. C. M. CHAPMAN.—I would like to ask if they cannot give us a little more about this deterioration due to animal and vegetable oil? Mr. Pearson says that only two out of a number of animal or vegetable oils showed deteriorating effects on concrete, probably due to the richness and density of the mixing. Previous tests reported to the society have shown that practically all, I believe, of vegetable and animal oils swell and soften in ordinary mixtures. If it is true that the oil, when it does come in contact with an experimental mixture of cement and sand, softens it and swells it, is it not only a question of time when these same oils will soften and swell the interior of a storage plant, or will the richness of the mixture and density, and therefore the depth of the layer that the oil acts upon in a given time, produce such a small layer that the action will be slowed up? Is it not just a question of time?

Mr. Pearson. MR. J. C. PEARSON.—We expected more signs of deterioration than we found. Outside of those two cases, there is absolutely no evidence where the qualitative methods we employed indicated any injury whatever. We looked for it very carefully in the case of the linseed oil tests, thinking that since we would expect saponification there, the concrete itself would show, but even in the 1:4 concrete ,it was not noticeable; it seemed as hard and firm as when originally made. I do not know what will happen in the future, but I do not believe any of the mineral oils will have any effect.

Mr. Freeman. MR. J. E. FREEMAN.—If I recall correctly, a series of tanks, each holding eight thousand gallons, was built by the Port of Seattle, located beneath one of their piers. Those tanks were built in 1917, and since then have been used for the storage of a number of different kinds of vegetable

oils. I understand that cocoanut oil is included in that list, and to date there has been no report from the engineer that these tanks are giving other than satisfactory service. Furthermore, some of the cottonseed oil manufacturers located in the Southern states and in Texas, have sumps or wells beneath their cottonseed presses where the oil goes before it is transferred into the storage tanks or tank cars. Some of these sumps are constructed of concrete, and so far as reports have been obtained from these companies, the use of concrete in this connection seems to be perfectly satisfactory. The oil going in and out has had no deteriorating effect upon the concrete. Mr. Freeman.

I would like to make a remark in connection with a point in Mr. Andrews' paper relating to the covering of the surface of the concrete tank with a sodium silicate solution, in order to permit the hardening of the concrete to proceed without being affected by the oil when that is put into the tank. I wonder whether it might not be possible that the advantage was more on account of the oils—the fuel oils used in this section of the country, as compared with the fuel oils used in the Western section, for example. Those oils in the Western section are of a lower gravity, running about 25 deg., or at least under 30 deg., Baumé, while I believe that the oils in this section are generally higher. I know of a number of cases of concrete tanks that have been built in the Western section and used for the storage of the lower gravity oils, where the tanks have simply been constructed with the proper mixture and proper method and then properly cured without the addition of any coating of this kind on the surface, and yet when the oil has been placed in the tank afterwards and the tank has been in service for several years, there has been no evidence of any appreciable penetration.

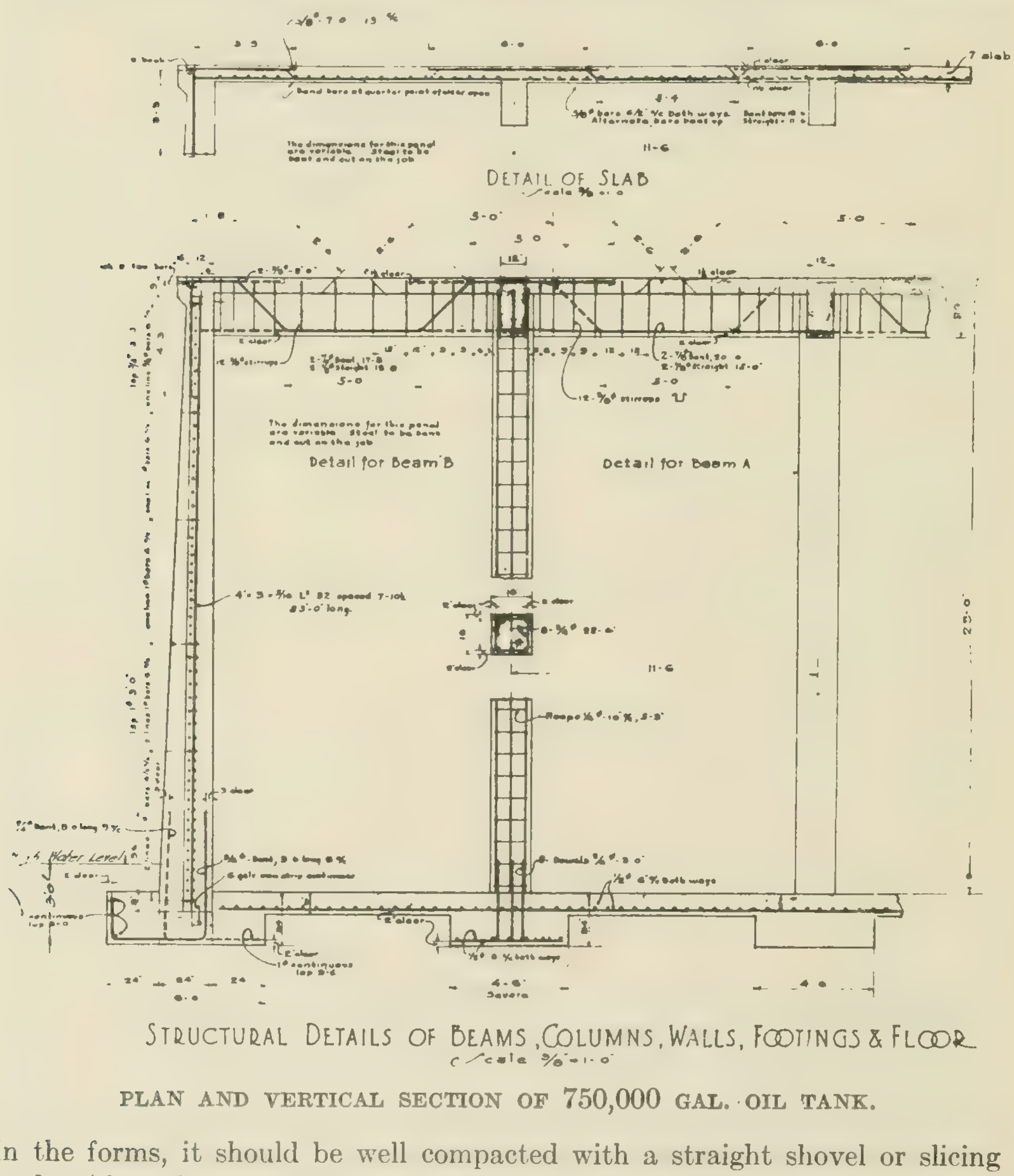
A. B. McDANIEL.—The writer has been much interested in the subject of fuel oil tanks, the design and construction of which the Construction Division of the Army have had under consideration during the past eighteen months. It has of course been necessary to provide tanks for the storage of fuel oil and gasoline at the various camps, posts, depots, terminals, warehouses and other projects in order to have on hand for immediate use fuel oil and gasoline for internal combustion machinery used in motor cars, trucks and airplanes. Mr. McDaniel.

Until October, 1918, steel was used almost entirely in the construction of storage tanks for fuel oil and gasoline. However, with the embargo placed on steel by the War Industries Board, it became necessary to utilize some other material, and the Construction Division immediately prepared some plans and specifications for the construction of reinforced concrete storage tanks of varying capacity. In this preliminary work, the engineers of the Bureau of Standards generously co-operated with this division and furnished much valuable information concerning the work which they had done in the design, construction and testing of fuel oil tanks.

The accompanying drawing gives the design which this office prepared for gasoline or oil tanks of 750,000-gal. capacity. Others prepared for capacities of 12,000 gal. and 250,000 gal. are not shown here. The following are the specifications which were prepared and adopted for the construction of these tanks.

"Concrete used for tanks should consist of one part cement, two parts

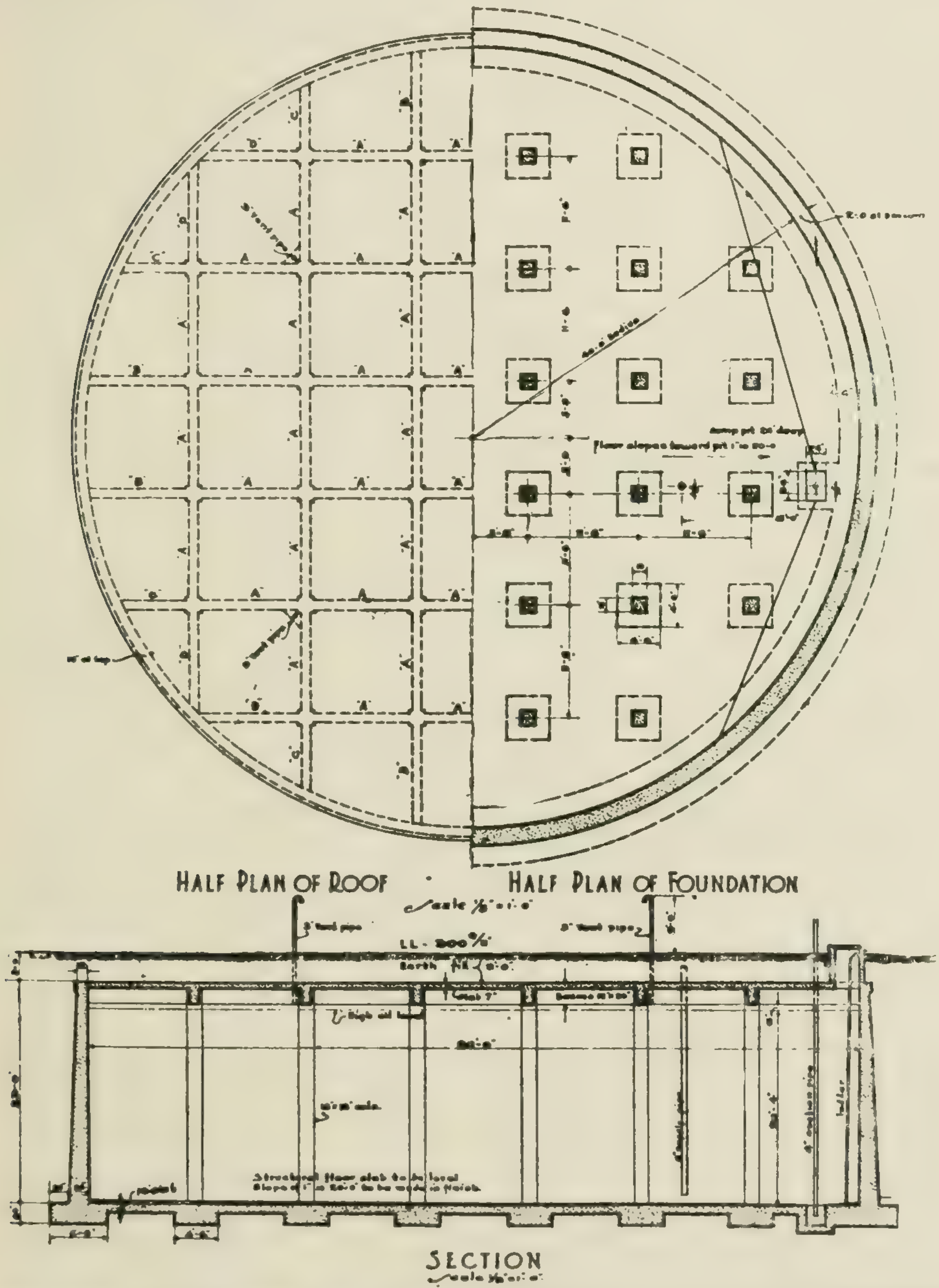
Mr. McDaniel. sand, three parts broken stone or gravel, and 10 lb. of hydrated lime for every 100 lb. of cement used. Crushed stone or gravel used, should be such as will pass a 1-in. ring. The materials should be mixed together in a batch mixer, at least 2 minutes, after all the ingredients are in the mixer. The concrete mixture should be of such consistency, as will flow sluggishly into the forms and such as can be conveyed from the mixer to the forms, without separation of the coarse aggregates from the mortar. While being deposited



PLAN AND VERTICAL SECTION OF 750,000 GAL. OIL TANK.

in the forms, it should be well compacted with a straight shovel or slicing tool. After the process of pouring concrete is begun, it must proceed as rapidly as possible, thus reducing the number of laitance seams. Where it is necessary to bond new concrete to old, the old concrete must be thoroughly cleaned, roughed, drenched with water, and then slushed with a mortar consisting of one part cement and one part sand. The entire bottom of the tank must be troweled perfectly smooth. The interior surfaces of the side walls and the surfaces of all the columns should be made smooth, by careful

and continued use of the slicing tool, during the process of placing the concrete. **Mr. McDaniel.** Any rough spots which show after the forms are removed from the interior, should be made smooth by mortar applied with a trowel. Forms should be



DETAILS OF 750,000 GAL. FUEL OIL TANK DESIGNED BY CONSTRUCTION DIVISION OF THE ARMY.

of dressed and matched lumber and carefully joined to insure smooth surfaces. Forms should be left in place at least seven days."

All of the above applies to tanks for the storage of fuel oil only. If gasoline is to be stored, the following additional instructions apply:

Mr. McDaniel. "When thoroughly dry, the entire interior surface of the bottom, side walls, and columns, should be sprayed with a coat of the best "long-oil" spar varnish thinned with 20 per cent volatile mineral spirits and applied under a pressure of 60 lb. per sq. in. by a paint gun. After this coat has dried for a period of at least 24 hours, it should be followed by two more coats, separated by an interval of not less than 48 hours, using pure undiluted spar varnish and applied in the same manner. Each varnish coating should be put on in such a thickness, that one gallon will cover approximately 200 sq. ft. The volatile mineral spirits should be a hydrocarbon distillate, water white, neutral, clear and free from water. It should have no darkening effect when mixed with basic carbonate white lead. The varnish should comply with the War Department Specification No. 6, with particular attention to the water tests.

"As an alternate to the instructions of the preceding paragraph, it may be deemed advisable in some cases to use one of the proprietary proofing methods of the market, but, only on condition that the concern awarded the contract, shall furnish an acceptable surety bond guaranteeing the construction to be proof against leakage for a given period of years."

The construction of fuel oil and gasoline tanks was started early in November, 1918, at several camps and depots, largely at Motor Transport Repair Stations. The signing of the armistice on November 11 stopped the larger part of this work, and in most cases the work has not been completed to date. This office has been able to secure very little information in regard to the construction and use of the tanks which were completed and are now in service, and such information as is available is rather incomplete. However, the following items may be of interest.

At Camp Normoyle, San Antonio, Texas, a 100,000-gal. capacity fuel oil tank was completed on July 1, 1919 at a cost of $7,000. The purpose of the tank is to supply fuel oil to boilers. The tank has a length of 75 ft., a width of 16 ft. 4 in. and a clear depth of 14 ft. 7 in. The walls are 18 in. thick at the floor line and 12 in. at the top, except in the partition walls which are 12 in. throughout. The tank is divided into four compartments of 25,000 gal. each. The present condition of the tank is excellent and there is no evidence of cracks or voids.

A 12,000-gal. capacity gasoline storage tank was constructed at Camp Knox, Stithton, Ky., for the use of the Motor Corps Service Station. The tank is 18 ft. inside diameter and has a depth of 7 ft. The cost of the tank was $1,598.03.

The experience of this division shows that it would be advisable to coat with asphalt the outside of all reinforced-concrete tanks buried in the ground. This coating should be applied as soon as possible after the exterior wall forms are removed, so as to allow the wall to dry out sufficiently before the interior coating of spar varnish is applied.

The experience of the Construction Division clearly indicates that the subject of reinforced-concrete fuel oil and gasoline tanks is a splendid field for further study and investigation. It is hoped that during the coming year considerable data will be secured which will be of value and interest to the profession.

SOME REMARKS ON EARTHQUAKE RESISTING CONSTRUCTION IN CENTRAL AMERICA.

By Juan I. de Jongh.*

Earthquakes in the Republic of Guatemala during the years 1917 and 1918 have given the writer an unusual opportunity to observe the resistance of old and modern buildings in Central America.

It may not be out of place to begin by noting that the earth movements, where any earthquakes occur, move differently, according to the kind of earthquake. The following is a list of the kind of earthquakes: (1) Violent eruptions of volcanoes; (2) subterranean explosions of unknown substances; (3) the caving in of some part of cooled off subterranean earth crust; (4) the sliding down of part of the surface soil resting on certain formations of rock, but undermined between the rock and the surface soil by water.

The first kind produces a horizontal movement and if of any duration or strength is the most fatal to building construction. The second kind produces an up-and-down movement, very similar to the movement of sea waves. The third produces more noise than the surface movement and may sometimes produce the lowering of the surface in the district where it occurs. The fourth—a sliding movement—is entirely local, but very dangerous to any buildings on the part moving, and to buildings at the foot of the hill in the immediate neighborhood.

There is another movement, often wrongly called earthquakes, which is an atmospheric pressure on the roofs of buildings. This pressure has its origin in the electric explosions in the upper atmosphere, and sometimes produces a crackling noise and rattling of doors and windows, but does not have any perceptible structural effect, nor is it strong enough to move the earth's surface. It may not be out of place to mention here that during the writer's forty years of climatological and earthquake studies in Central America he is positive that no earthquakes have had any influence on the atmosphere, as is sometimes maintained by other writers.

Another very important fact, and one which has the greatest consequence in the destruction of the place where earthquakes occur, is that none of the above-named movements occur in straight lines, but run in very broken curves and circles, according to the formation of the under strata and their resistance. This is the reason why, after a strong earthquake, some buildings and whole streets in the same town remain standing; whereas other buildings built of the same material, under the same conditions, are thrown down.

It is safe to divide the damage done by earthquakes into four parts, as follows: (1) Damage done by the earthquake; (2) damage done

* Guatemala City, Guatemala.

through unskilled and unscientific construction; (3) damage done because of decayed state of timbers and bad material; (4) intentional and criminal damage done by inhabitants.

Is there any kind of construction known to date that can withstand a heavy earthquake of the intensity of 12, as calculated by the Cancassi scale of seismic intensity, 12 signifying the acceleration measured in millimeters per sec. and indicating the most serious shock? The writer's answer is that during the years he has made a study of earthquakes and their consequences no such seismic intensity has happened in Central America, and if it has in any other part of the world, all buildings would have been destroyed. Through his observations, however, the writer maintains that reinforced-concrete construction will resist the strongest earthquakes that have occurred.

Much of the old way of building in Central America was, and is still, of sun-dried bricks of different sizes, but most 16, 9 and 4-in., called adobes. There are still ways in existence 33 to 49 in. thick. Generally the walls of ordinary houses or buildings, except church walls, are from 12 ft. to 15 ft. high, the foundations usually being made either of round river stones or decayed rock set in well-tamped clay. In some cases wild cane or bamboo was inserted lengthwise in the layers between the adobes or sun-dried bricks. At present this same way of building houses of dried bricks is largely used by natives in the country as well as in towns, not because there is a general belief that this kind of construction is more healthful or earthquake-proof but because it is the cheapest way of building.

These adobe houses are not earthquake-proof and are not elastic enough, when once out of perpendicular, to return to their former perpendicular shape, and, as experience has shown, are the first to succumb to earthquakes. If they are not actually shaken down they are generally split in different parts. Another reason why this kind of building fails is because the wall plates are laid flush with the inside of the wall. This is why one often sees the roof fallen inside the building, and the walls, however much damaged, still standing; the earthquake movements made the roof slide off the walls. Also the adobes have very little resistance when a crushing weight is applied.

Another type of building formerly used a good deal consisted of walls built between two heavy boards and constructed of either round stones, large, hard broken pieces of stone, and sometimes, but in rare cases, of rough cut soft stone for facings on both sides of the walls, the spaces between being filled up with any kind of small stone and lime mortar. In some cases the filling was done with small stones and tamped clay. This kind of wall has also not been able to stand earthquake movements.

Still another way of building is called "Bajareeque." This consists of hardwood posts buried in the ground from 4 to 5 ft. to the wall plates and 4 to 6 ft. apart, the space between these posts being generally filled in with tamped clay mixed with small broken pieces of stone or broken tiles,

kept in position by wild cane or bamboo, secured, in older times, to the post by rawhide lacings, but at present are fastened with nails. This kind of construction has withstood earthquakes better than any other native construction.

Ordinary brick have also been used, but this in general has proved very unsatisfactory in earthquake movements, not because a good and properly made brick building does not stand the earthquake movements but because, in Central and South America, poor brickwork is done. The writer knows of a two-story brick building, erected in 1888, which, since that time, has stood many earthquakes without being damaged. However, the walls of the building were properly anchored to the timbers and in the corners. The reason brick walls in South America generally fail is because poorly made bricks are used: too much lime mortar is used between the bricks; the bricks are not wet enough; no bond or anchor is used, and last, but not least, the natives are very inferior bricklayers. The walls, also, are usually made too thin for resisting the earthquake movements. In general, brick buildings cannot be called earthquake-proof.

Still another type is the wood-frame building. Houses thus built, if properly framed as originally intended, have proved satisfactory in earthquake countries, though there are several cases where, in heavy earthquakes, whole houses were completely overturned, and other instances where they were only partly destroyed. One great objection to native made timber houses is the attraction they have for vermin. There are cases where such houses have become uninhabitable because of vermin in less than five years and have to be burned. Another objection to native wood-frame house is that the native wood does not last for any length of time, and if hardwood is used for the outside and inside linings the tropical climate makes it either split or become out of shape. If the posts are made of hard, durable wood no nails will enter, and if not ventilated it will decay within a very few years. Foreign lumber lasts longer if of good quality, but if it is not painted every year (and no one seems to do so) it will last but two or three years. The above remarks result from observations in Central American tropical climates.

Taken all in all, wood houses, if properly framed and not simply nailed together, are adaptable for earthquake countries, especially in the country sections, but should be prohibited in towns, where vermin abounds, and also because of the fire risk.

Another newly adopted way of building is iron-frame wall houses and roofs, lined with expanded metal sheets and afterwards plastered with lime or cement mortar. Two small buildings of this type, erected on the Pacific coast of Guatemala, have stood the earthquake test first-rate, the plastering not showing any cracks. At present several such buildings are under way in the town of Guatemala. The construction for the frame is of metal lumber T-joist, with channel studs joined by screw bolts. The disadvantages of the wood-frame houses are thus done away with, the only lumber used being for doors and windows and a few strips of wood to receive the

metal ceiling. The roof is made of the same material, plastered, after the principal lime-mortar coat is put on, with pure cement put on dry on the moist lime plaster, thus making a very satisfactory waterproof roofing and having the advantage of being non-heating, also of not causing an unpleasant noise, as is the case when it rains on a corrugated or metallic roof. This kind of construction is recommended for earthquake countries.

Another method of construction, the modern way, is the reinforced-concrete building and other construction in concrete. The first reinforced-concrete building in Central America was put up in Port Lemon, Costa Rica, about fifteen years ago by the Hennebique Construction Co., of New York, using the well-known Hennebique system. The next country in Central America to adopt the reinforced-concrete construction was San Salvadore. The first reinforced-concrete building was erected here in 1910-1911. Since that date reinforced-concrete construction has become quite general.

The first reinforced-concrete semi-detached laborers' cottages erected in Guatemala were built in 1913, and the first two-story reinforced-concrete building was erected in the city of Guatemala in 1916-1917. On the Atlantic coast several concrete buildings have been erected since 1915.

A new system of reinforcement, devised by the writer, was used for the first time in concrete construction, in the building referred to in Salvadore. Reinforcement consists of the abundant use of triangular mesh woven wire wired to iron joists placed at certain distances perpendicularly in the concrete walls. These joists or posts are bolted to the foundations and extend to the roof of the building. All of these posts are termed "lumber metal joists," and their width is in proportion to the thickness of the concrete walls. Four inches apart holes of ½-in. diameter are made in the joists through which the wiring passes to the triangular mesh. The mesh is cut in lengths to fit between the posts. The thickness of the concrete walls was in all cases calculated to cover the posts from ¾ to 1¼ in. The flat roof of the building is also constructed of concrete, the triangular mesh being used for the reinforcement.

In all the concrete buildings known to the writer in Central America, including the ones more recently erected in Tela, Honduras, only one serious failure has been reported. This was in the city of San Salvadore—a two-story house, of large dimensions, the ground floor of which is used for wholesale commercial business, the top floor as a residence. The reason of its failure is that the contract was placed at too low a figure and was built by the same party that submitted the plans—no architect or engineer being employed by the owner.

In all of the places herein mentioned very strong earthquakes have occurred in the past few years, principally in Salvadore, where heavy earthquakes have been more frequent than in any of the other five republics. Properly reinforced-concrete buildings have shown more earthquake-proof qualities than any other method of building.

Summarizing the above observations and results of the different

systems of building, the following alternating constructions for earthquake countries are listed in order of their merit: (1) Reinforced-concrete construction when financial circumstances permit; (2) iron frame buildings with expanded metal ribs; (3) Bajareeque construction as above described, or posts with expanded metal with ribs in them with filling of tamped clay; (4) wood-frame houses, properly framed. lined either with expanded metal or wood.

In closing, it may not be out of place to say a few words about the force generated to produce some of the earthquakes, especially the kind described as No. 2. Taking, for example, the Guatemala earthquake of 1917-18, together with personal data, the strongest upheaval of the earth in the town of Guatemala occurred at 11.16 P. M. on Dec. 25, 1917. At that time the upheaving was 1¾ in., the earth moving like the waves of the sea for about ten seconds. This movement covered a radius of forty miles, the center of the disturbance being near the city. Taking the approximate depth of the earth crust at about 35,000 ft. of solid rock and a radius of 40 miles, the force required to lift this mass the distance above mentioned would be 151.962 million h.p.

LAYOUT AND EQUIPMENT OF THE GOVERNMENT CONCRETE SHIPYARDS.

By A. L. Bush.*

The United States Shipping Board, Emergency Fleet Corporation, adopted its program for constructing concrete ships at a time when speed in construction was considered one of the most important factors. In order to avoid climatic conditions which would be apt to cause considerable delay or extra expense during a portion of the year, it was decided to establish the concrete shipyards, as far as practicable, in sections of the United States where the climate is such that work can be carried on throughout the entire year. On account of the work being done in other yards, it was also decided that the locations must be such as not to interfere with established steel or wood shipyards, either in labor or in construction equipment.

In investigating sites, therefore, special attention was paid to those located on the South Atlantic and Gulf Coasts and on the Southern Pacific Coast. In the selection of the sites particular attention was paid to each of the following, viz., the availability of site, with arrangement for transferring title to another in case the Emergency Fleet Corporation decided to discontinue building ships on them; the location of site with respect to protection from storms, hurricanes, tidal waves or excessive rise of rivers; the accessibility of the site; the depth of water, width of channel, together with amount of dredging necessary; the transportation facilities—both freight and passenger; the sanitary conditions; the housing facilities; the labor market in the vicinity; the possible future use of the site; the availability of lumber, steel, cement and sand, without expensive freight; the size of site, an attempt being made to secure at least 2500 ft. frontage with 1200 ft. depth; the character of the soil, a firm sandy soil being preferred, care being taken to avoid the necessity for excessively long piles for ways and bulkheads; and the availability of power and water.

Five Yards Built for Eight Ships Each.

As a result of the investigations, shipyards for the construction of concrete ships were built at Wilmington, N. C., Jacksonville, Fla., Mobile, Ala., Oakland, Calif., and San Diego, Calif. In addition to these five yards, one small wood shipbuilding way was leased from the Brunswick Marine Construction Co. at Brunswick, Ga., on which the experimental 3000-ton cargo ship "Atlantus" was built. At North Beach, Long Island, the 3500-ton cargo ship "Polias" was built by the Fougner Concrete Shipbuilding Co. in their yard.

The five yards mentioned above were all planned with the assumption

* Head, Construction Branch, Concrete Ship Section, U. S. Emergency Fleet Corporation, Philadelphia, Pa.

that at least eight ships should be built at each yard. The original contracts at each of the five yards call for eight ships with a probable increase in the number. When these yards were being planned it was also the intention to outfit the ships at the yards where the hull was built. Since then the plans were changed so that no outfitting will be done either at the Wilmington or at the Jacksonville yards, but outfitting is being done at each of the other three yards. The hulls being built at the Wilmington and at the Jacksonville yards will be outfitted by the Jacksonville Ship Outfitting Co., at Jacksonville, Fla.

Each yard was planned to have at least four shipways, the idea being that at least two ships would be built on each way. The curtailment orders issued, after the armistice was declared, reduced the number of ways to be completed to two in each yard, further work on the other ways was discontinued. The contemplated yard layout for each yard, as indicated on the accompanying diagrams was made according to the plans of the superintendents. The general layout of the yards was approved by the Concrete Ship Section of the Ship Construction Division of the Emergency Fleet Corporation; and the details of construction of the yards and buildings were approved by the Section of Concrete Shipyard of the Shipyard Plants Division of the Emergency Fleet Corporation. Numerous changes were made in the original plans of the superintendents at the suggestion of the home office of the Emergency Fleet Corporation.

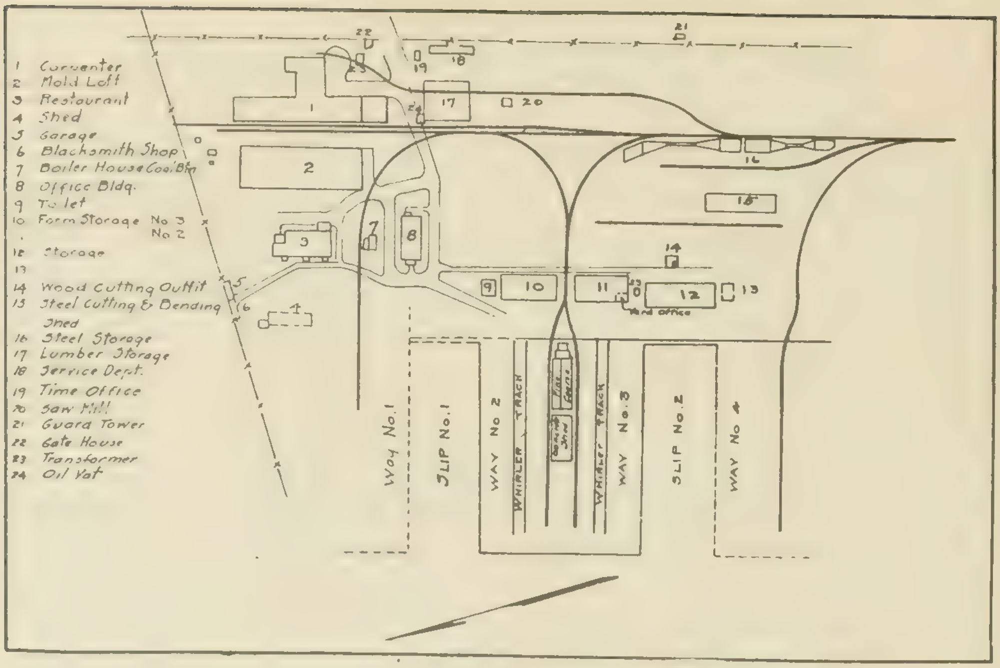
SIDE LAUNCHING ADOPTED.

Before any satisfactory planning of yards could be done it was necessary to decide whether end launching or side launching would be adopted. The method of launching has a decided bearing on the cost and future utility of the shipyard, the ease of ship construction, and the launching stresses. These subjects were all kept in mind during the planning of the yards.

The subject of the necessary cost of the entire yards does not lend itself to any simple statement as to which of the two methods of launching will produce the cheaper yard. The cost depends upon the amount of dredging, the width of slip into which the ship is to be launched, the length of waterfront available for ways and many other items, which must be taken into account. When salvage value is considered, however, it raises a strong point in favor of some layout which will fit the property for other use than that of ship construction. The layout for side launching with the ways arranged on piers with slips at the sides is probably the most adaptable for future use, as after ship construction is completed there will remain a dock and bulkhead immediately available for general use. In end launching, no such valuable waterfront property remains. This work being done under war conditions, it was quite probable that the cost would be greater than in peace time and, therefore, this extra salvage value, which outweighs the difference in cost, was an item of much importance.

WHY SIDE LAUNCHING IS BETTER.

There is a decided advantage in favor of side launching with regard to the ease of ship construction, especially in case of concrete ships. In end



OFFICE

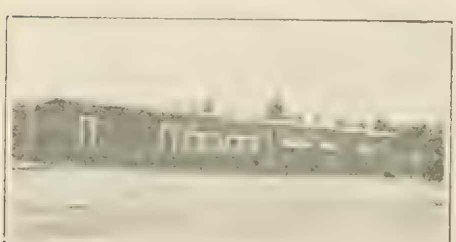
STOREHOUSE

RESTAURANT

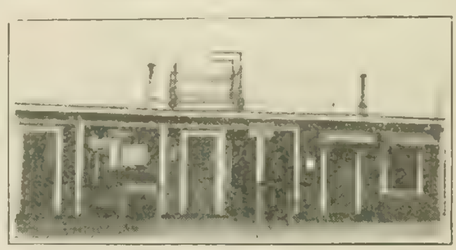
CEMENT SHED

MOLD LOFT

STEEL BENDING SHED



EMPLOYMENT OFFICE



TIME OFFICE

FIG. 1.—LAYOUT AND VIEWS OF CONCRETE SHIPYARD AT WILMINGTON, N. C.

launching the ways are given a slope of from $\frac{5}{8}$ in. to $\frac{3}{4}$ in. to the foot, and the ship is built practically normal to the ways. It follows that the location of all horizontal and vertical members must be determined with reference to this slope. In concrete ships the placing of frame reinforcement and the setting of frame forms are just two instances of the effect of this slope on the ease of concrete ship construction. In both of these cases the deviation from the vertical necessitated by the slope of the launching ways adds tremendously to the detail of construction. It has also been found that the keelson reinforcing steel is apt to creep toward the stern in end launching, necessitating some method of anchoring the top steel so as to avoid this creeping.

One point in favor of end launching is the small amount of outside staging which it would be necessary to remove during launching. In concrete ship construction, this staging, acting as it does to support the forms and to take up the lateral stresses of the plastic concrete, will be more complicated than the outside staging for steel ship construction, for in the latter case the chief function of the staging is to act as a scaffolding for the workmen. If a ship is to be launched sidewise, all of the outboard and possibly some of the inboard staging must be removed, whereas in an endwise launching, only the staging aft of the midship section is in the path of the ship as it leaves the ways. Aside from these construction details, the actual launching operation is simpler for sidewise than for end launching.

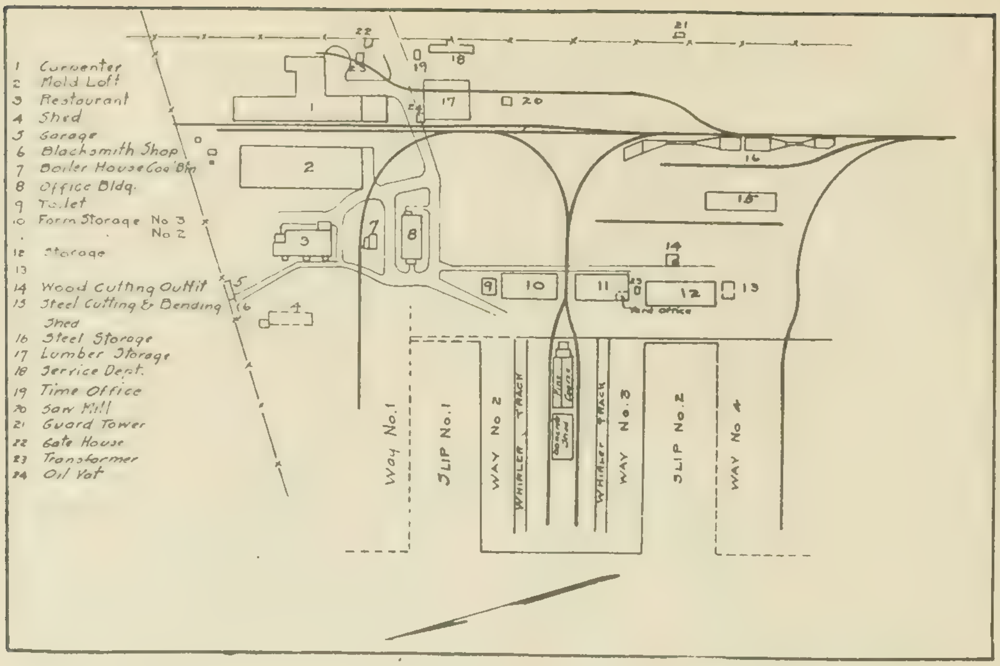
LAUNCHING STRESSES INDETERMINATE.

Very little was known regarding the stresses developed in launching at the time the decision as to the types of ways was made. Theoretically no stress set up in the ship is as great as the stress experienced when in the seaway On this basis, launching stresses are relatively unimportant for the steel ship, as at the time of launching it can contain all its strength members, and will, therefore, be as strong as it ever will be. A concrete ship, on the other hand, is weakest immediately after pouring and from then on grows in strength as the concrete ages; and has not reached anything like its maximum strength at the time it is launched.

Computations indicate that the stresses developed in end launching are higher than any stresses developed in the ship when in service except when in rough seas. The stresses for side launching are believed to be much less than for end launching. It follows that with side launching concrete ships could be put into the water earlier than with end launching. This was an argument which carried great weight during the war when speed in construction was the controlling factor in all shipbuilding contracts.

The weight of the hull was also taken into consideration in deciding the method of launching. This is more indefinite than in steel vessels. The question of aggregate had not been settled at that time, and, therefore, the weight of the concrete was problematical. On account of this it was impossible to compute, exactly, the buoyancy of the vessel at launching time.

A plant arranged for side launching is more flexible than one arranged for end launching. If the ways are built to accommodate medium-sized



OFFICE

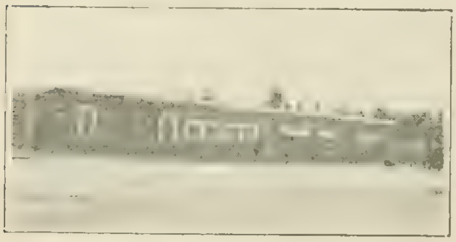
STOREHOUSE

RESTAURANT

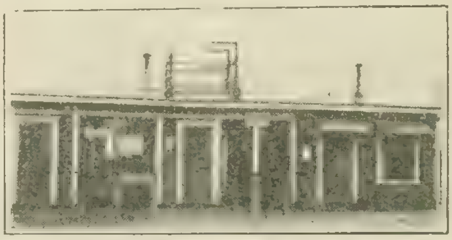
CEMENT SHED

MOLD LOFT

STEEL BENDING SHED



EMPLOYMENT OFFICE



TIME OFFICE

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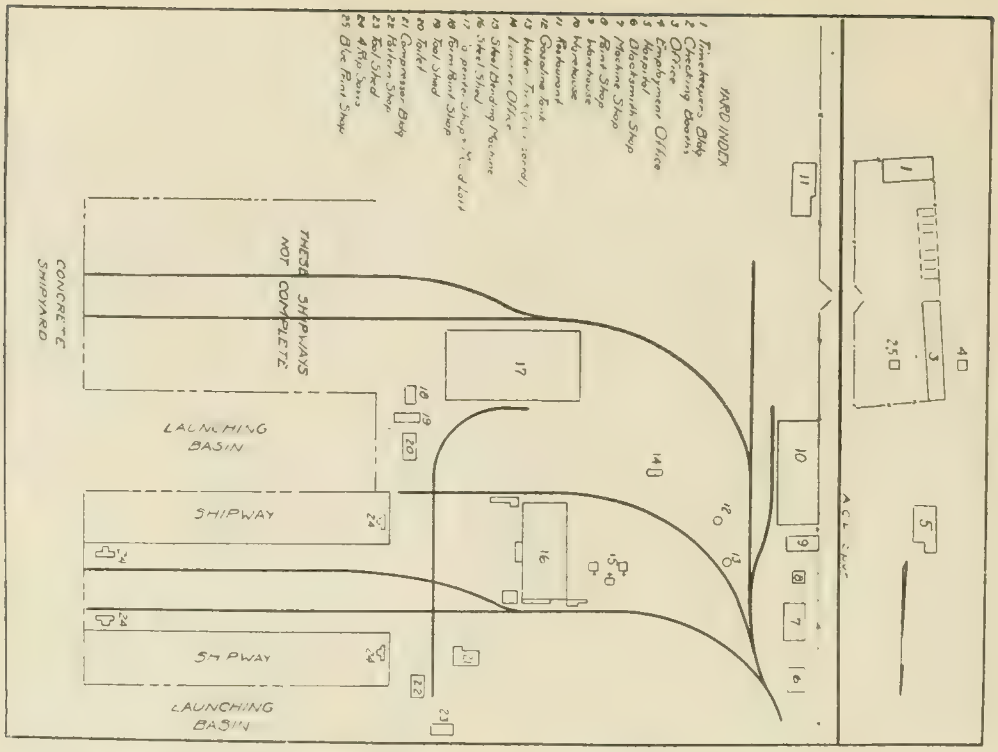
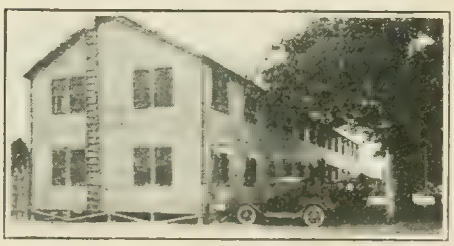
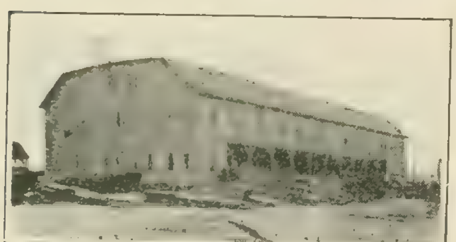




OFFICE

RESTAURANT



MOLD LOFT AND CARPENTER SHOP

STEEL SHED

MACHINE SHOP

WAREHOUSE

TIME OFFICE

CHECKING BOOTHS

FIG. 2.—LAYOUT AND VIEWS OF CONCRETE SHIPYARD AT JACKSONVILLE, FLA.

vessels, especially when arranged to build two vessels tandam, several small vessels, such as tugs, barges, etc., can be built at one time; while if designed for end launching this cannot be done without considerable change in plans.

While the majority of shipyards are arranged for end launching, a large number of modern plants are arranged for side launching, this is notably the case with shipyards on the Great Lakes.

The area necessary to be covered with piling to support the ship during the construction of the hull is the same whether side or end launching is used. If side launching is used, it is necessary to bulkhead the slip and the pier, while if end launching is used it will be necessary to pile an additional area to support the outboard launching ways which area will be approximately equal to one-half that required to support the hull while under construction.

Briefly summarizing, in a general way, the points in favor of each of the two methods of launching, we find that for end launching, the first cost of the ways is probably slightly less, and that in order to launch the ship it is necessary to remove staging only aft of the parallel middle body instead of for one entire side. On the other hand, for side launching the ways constitute a permanent improvement to water front property and therefore has a greater salvage value; greater convenience in construction of hulls since they can be built on a horizontal keel, and lower launching stresses.

PILING WAS STANDARDIZED.

The shipways in a concrete shipyard differ somewhat from those of a steel shipyard. The piling is arranged differently, owing to the fact that the weight of the hull is distributed over the entire area of the ship. The size of the ways as built is about 78 x 460 ft. or large enough to provide space for the construction of 7500-ton dead-weight capacity concrete hulls. It was necessary to pile this entire area, as the outboard portion had to be made capable of supporting the ship while launching, and the piling on both sides of the hull had to support staging which was of sufficient strength to support the concrete while in a plastic state. The assumed load on each pile was 15 tons and on account of the weight of the concrete hulls, the concentration of loads during construction and during the blocking and launching of the hull, it was found necessary to space the piles about 4 ft. centers longitudinally of the hull and 6 ft. transversely. The area of the ways is somewhat larger than would be required for steel ships of the same capacity. The launching load increases the pressure on the outboard piling and it was found necessary to double the number of piles in the outer row.

An investigation was made of conditions at each yard to determine the kind of piling to be used. The results of these investigations showed considerable difference as to local conditions. These differences are indicated by the kind of construction used at the various yards. At Wilmington, N. C., the ways were built of green piling, and instead of a sheetpile bulkhead, as originally designed, a cribbing was substituted. At Jacksonville, Fla., about 17 per cent of the piling was creosoted and the remainder was green, while the bulkheading was made of creosoted sheet piling. At Mobile, Ala., green piling was used for the ways with a bulkheading similar to that at Jacksonville.

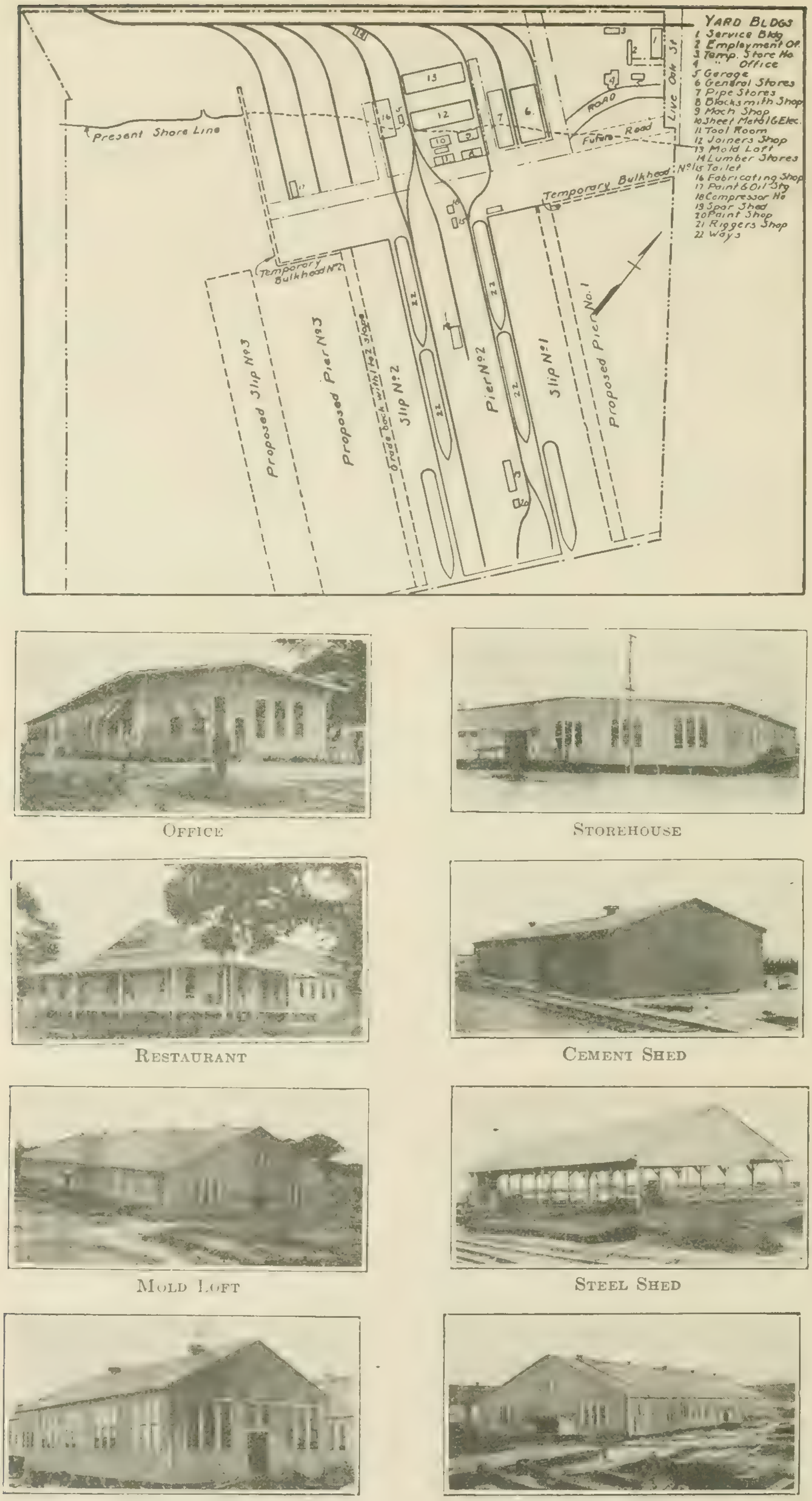


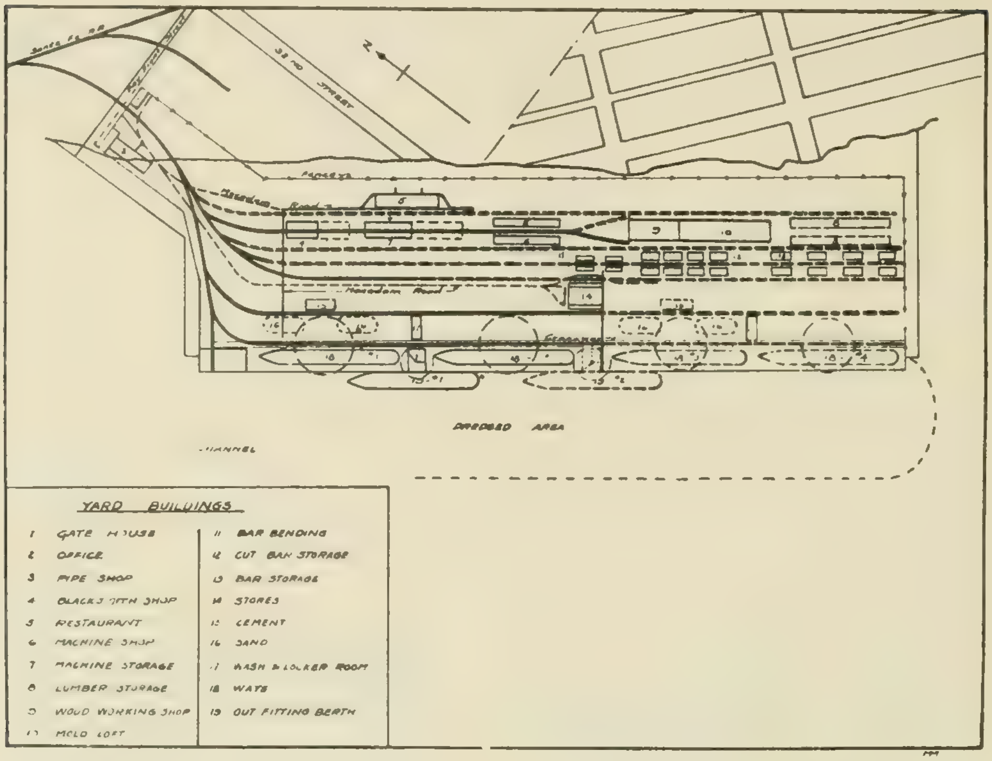
FIG. 3.—LAYOUT AND VIEWS OF CONCRETE SHIPYARD AT MOBILE, ALA.

At San Diego, Calif., green piling was used with a reinforced-concrete sheet piling. At San Francisco about 30 per cent of the piling was creosoted and the balance green. No sheetpiling nor cribbing was used.

MUCH DREDGING REQUIRED.

In the design of various yards, it was decided that it was necessary to have a minimum depth of water of $17\frac{1}{2}$ ft. and therefore the launching slips were in all cases dredged so as to provide this depth at low tide. At Wilmington, the amount of dredging actually done amounted to about 315,000 cu. yd., composed chiefly of sand but containing a large amount of sunken logs and stumps which made the dredging problem a very serious one. At Jacksonville, the amount of dredging done was about 290,000 cu. yd. The material dredged consisted of sand and clay, partly solidified by concretion and containing some sunken logs. At Mobile, the quantity was about 600,000 cu. yd. and the material consisted chiefly of soft material with a few spots of hard. At San Diego, the total amount dredged was about 400,000 cu. yd. while the material here was very hard, and difficulty was experienced in dredging it. At Oakland, the quantity was about 85,000 cu. yd.; this amount being small on account of the nature of the land upon which the yard was located, the site having been made by a former dredging process; and the material was composed chiefly of a soft material. Had these yards all been completed as planned the amount of dredging would in some cases have been considerably greater than the amount actually dredged.

The number and kind of buildings varies in these yards. In general, however, the number of buildings required for a concrete shipyard is approximately the same as required for a steel shipyard. Each of the Government concrete shipyards were provided with an office building, general storehouse, a restaurant or lunch room, a carpenter or joiner shop and cement sheds. A mold loft was erected at each of the yards excepting at Oakland, where the floor of another building was temporarily used for a mold loft. Each yard also was provided with a machine shop, excepting the one at Wilmington, where the machine shop was leased during the time the shop was required. At Wilmington and Jacksonville, buildings were provided for form and pattern storage, while at the other yards the forms were either stored in portions of buildings used for other purposes or were stored in the open. At Wilmington and Oakland, buildings were provided for saw-mills, while the other yards have all the sawmill equipment found necessary at those yards, housed in the carpenter shop or have machines set up at convenient places in the yards, in which cases, the machines are protected by being roofed over. At Jacksonville and Oakland, hospital buildings have been provided while at the other yards, the hospital is located either in a portion of the office building, as at Mobile, or in some other convenient buildings. Tool rooms are provided in all yards; in some cases, buildings are erected for this purpose while in others, portions of other buildings are used. Paint storage shops have been built at the various yards and are small one-story buildings in each case. Pipe shops have been built at Mobile, San Diego and Oakland and are one-story buildings of about 5000 sq. ft. floor area.



OFFICE BUILDING

STOREHOUSE

RESTAURANT

CEMENT SHED AND FIELD OFFICE

MOLD LOFT AND WOODWORKING SHOP

MACHINE SHOP

FIG. 4.—LAYOUT AND VIEWS OF CONCRETE SHIPYARD AT SAN DIEGO, CAL.

Buildings for blacksmith shops have been built at Jacksonville, Mobile, San Diego and at Oakland. Other buildings such as storage sheds for finished lumber, gate house, time office, employment building and other small structures were built wherever they were considered essential. There is, however, no uniformity in this matter.

DETAILS OF VARIOUS BUILDINGS.

The mold loft at Wilmington is a one-story building $96\frac{1}{2}$ x 262 ft.; at Jacksonville, it is on the second floor of a two-story building 85 x 200 ft.; at Mobile, it is a one-story building, 76 x 240 ft., and at San Diego, it is ao ne-story building, the total size of which is 60 x 440 ft. only a portion of which is used for mold loft. All of these buildings are wood frame construction.

The office building at Wilmington is a two-story building 49 x 105 ft.; at Jacksonville, it is a two-story building 30 x 160 ft.; at Mobile, it is a one-story building, 45 x 110 ft.; at San Diego, it is a two-story building 48 x 140 ft., and at Oakland, it is a one-story U-shaped building, 44 x 94 ft. with wings 21 x $74\frac{1}{2}$ ft. and 21 x 64 ft. All of these office buildings are wood frame construction. Those at Wilmington, Mobile and Oakland are finished with plaster board and battons; the one at San Diego is plastered with lime mortar, and the one at Jacksonville has the studding exposed.

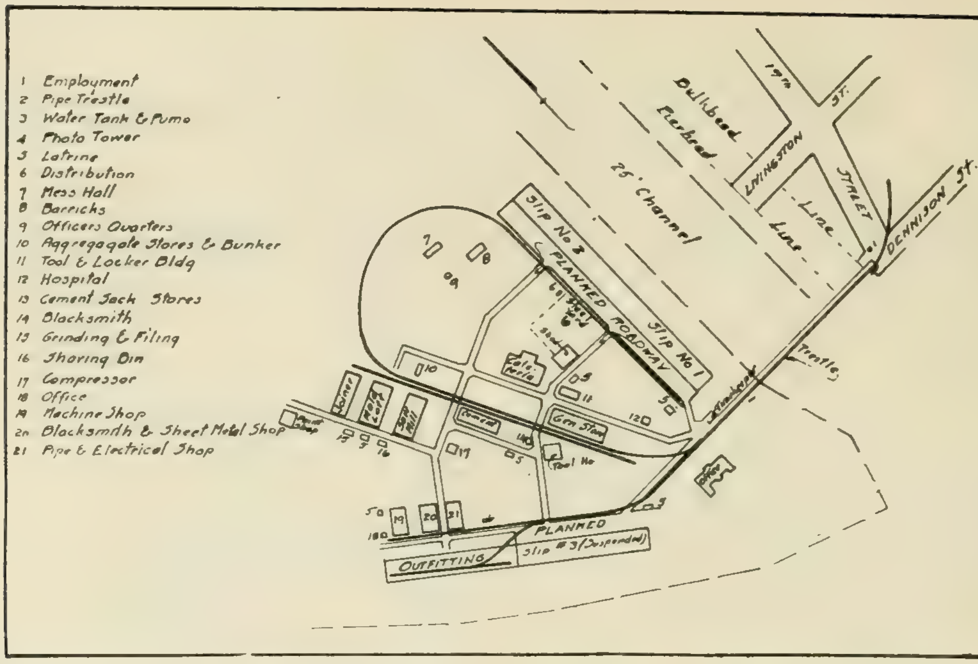
The general storehouse at Wilmington is a one-story building 48 x 150 ft.; at Jacksonville, it consists of two parts, a new one-story building, 60 x 111 ft. and an old two-story studio building 57 x 75 ft.; at Mobile, it is a one-story building 100 x 208 ft.; at San Diego, it is a one-story building, 60 x 100 ft., and at Oakland, a one-story building 60 x 200 ft. These buildings are all ordinary wood frame construction.

Steel fabricating shops were erected at four of the yards and consist of one-story open sheds with roof over the entire structure, and with dirt floors. At Wilmington, this shed is 40 x 70 ft.; at Jacksonville, it is 40 x 65 ft. and is used for steel storage; at Mobile, it is 61 x 154 ft.; and at Oakland, it is composed of two sheds each 43 x $54\frac{1}{2}$ ft.; and at San Diego, the machines are set outside with individual roofs over them.

At Wilmington, the restaurant is a building 60 x 102 ft., a portion being two stories high; at Jacksonville, it is a one-story building 30 x 92 ft. with a wing 12 x 52 ft.; at Mobile, it is a one-story building 58 x 67 ft.; at San Diego, it is a one-story building 48 x 176 ft. with a wing 22 x 59 ft., and at Oakland, it is a one-story building 75 x 138 ft. with a wing 29 x 58 ft.

The carpenter or joiner shop at Wilmington consists of a one-story building 60 x 260 ft. with a wing 80 x 100 ft.; at Jacksonville, it is the first story of a building whose second story is used for the mold loft and is 85 x 200 ft.; at Mobile, it is a one-story building 76 x 240 ft.; at San Diego, it is a portion of the building used for mold loft, a space about 60 x 160 ft. of this building being used for a carpenter shop, and at Oakland, it is a one-story building 50 x 150 ft.

The building for a machine shop at Jacksonville is a one-story building 30 x 60 ft.; at Mobile, it is a one-story building 40 x 80 ft.; at San Diego, a one-story building 65 x 125 ft. and at Oakland a one-story building 50 x 150 ft.



ADMINISTRATION BUILDING

GENERAL STORAGE

RESTAURANT

CEMENT STORAGE

MACHINE SHOP

BLACKSMITH SHOP

TIMEKEEPER'S OFFICE

BUREAU OF EMPLOYMENT

FIG. 5.—LAYOUT AND VIEWS OF CONCRETE SHIPYARD AT OAKLAND, CAL.

TABLE I.—MACHINERY IN THE FIVE GOVERNMENT CONCRETE SHIPYARDS.

Carpenter Shop	Wilmington	Jacksonville	Mobile	San Diego	Oakland
Swing cut-off saw	1	1	1	1	3
Band rip saw	1	1	1	..	2
Inside moulder	1	..	..	..	..
Outside moulder	1	..	..	..	..
Four-sided moulder	..	..	1	..	1
Pony planer	1	..	..	..	1
Joiner	1	2	..	1	..
Surfacer	..	1	1	..	1
Rip and dado saw	1	..	..	..	..
Post borers	2	..	..	1	..
Tenoner	1	..	1	..	1
Shaper	1	..	1	..	1
Band scroll saw	2	..	..	..	..
Degree band saw	1	1	..	1	..
Combination dado	1	.	..	..	1
Cut-off and rip saw	1	1	..	1	..
Belt sander	1	..	1	..	1
Boring machine	1	2	..	..	1
Plane knife grinder	1	1	..	..	1
Scroll saw filer	1	..	..	..	..
Band saw filer	1	2	..	..	..
Double emery grinder	2	..	..	1	..
Hand planer and joiner	..	1	..	..	..
Automatic band saw grinder	..	1	..	..	1
Cross cut saw	..	4	..	..	..
Circular saw filer	..	1	..	..	1
Straight edge ripper and joiner	..	..	1	..	..
Universal saw table	..	..	1	..	..
Wood lathe	..	..	1	..	1
Oil stone grinder	..	..	1	..	..
Tilting frame band saw	..	..	2	..	..
Mortisers	..	..	..	..	3
Sash stickers	..	..	..	..	1
Self-feed saw table	..	..	..	..	1
Four-side sticker	..	..	..	..	1
Arm sander	..	..	..	..	1
Band saw sitter	..	..	..	..	1
(Motor driven)	23	19	13	6	26

The pipe shops at the three yards where outfitting is to be done are all equipped to do the work well. The equipment is not elaborate but it is sufficient to enable the work to be economically done. The blacksmith shops at these yards are also equipped so as to provide for any work which may be required to be done.

In addition to the buildings mentioned above, form and pattern stores were built at Wilmington and at Jacksonville. At Wilmington, these stores

TABLE II.—MACHINE SHOP EQUIPMENT IN GOVERNMENT CONCRETE SHIP YARDS.

Machine Shop	Wilmington	Jacksonville	Mobile	San Diego	Oakland
Shaper	..	1	1	1	..
Power hacksaw	..	1	1	1	1
Drill presses	..	2	3	1	1
Power grindstone	..	1	1	..	..
Lathe	..	1	2	2	3
Bolt and pipe threading machine	..	1	..	2	1
Power emery grinders	..	2	1	2	..
Planer	..	1	1	..	1
Bull dog vices	..	1	..	..	..
Bench vices	..	4	..	..	..
Milling machine	..	..	1	..	..
Sensitive drill	..	..	1	..	..
Blast furnace	..	..	..	1	..
Radial drill	..	..	..	1	1
Combination punch and shear	..	..	..	1	..
Pipe bending table	..	..	..	1	..
Forges	..	..	..	4	..
Steam hammer	..	..	..	1	..
Hand traveling crane (5-ton)	..	..	..	..	1
Y. & T. triplex chain hoist (5-ton)	..	..	..	..	1
Sliding head drilling machine	..	..	..	..	1

consist of two one-story sheds with open sides, each building being 60 x 120 ft.; while at Jacksonville, a one-story building 20 x 28 ft. was considered sufficient. Cement storage sheds were built at each of the five yards and are of sufficient size to store cement for one 7500-ton hull. The saw-mill building at Wilmington is a one-story building 20 x 20 ft. with open sides; while at Oakland, it is a similar building 75 x 150 ft. Latrines are provided in each of the five yards, the buildings in all cases are one story in height and the size is governed

by local conditions. The hospital building at Jacksonville is an old two-story frame building 27 x 40 ft., formerly used for a dwelling; at Oakland, it is a new one-story building 29 x 33 ft.

TABLE III.—YARD EQUIPMENT IN GOVERNMENT CONCRETE SHIP YARDS.

Construction Equipment	Wilmington	Jacksonville	Mobile	San Diego	Oakland
Derrick.....	8 1-ton hand power, 1 2-ton 40-ft. boom stiff leg on flat car	8 2-ton 80-ft. boom stiff leg	....	1 15-ton 80-ft. boom	1 3-ton 74-ft. boom, 1 5-ton 90 ft. boom
Tower cranes	2 55-ft. tower whirler 85-ft. boom 3 ton at 75; 24-ft. gage track	....	....	2 67-ft. tower revolving cranes, 5 ton at 80 ft.	
Gantry......	....	....	2 10-ton 81-ft. span, 20 ft. overhangs each end 60-ft. hoist		
Locomotive Cranes			1 20-ton 50-ft. steel boom	1 20-ton 50-ft. boom	1 20-ton 50-ft boom
Yard Locomotives	1 35-ton	1 35-ton	1 40-ton	1 25-ton	1 42-ton
Flat cars and push cars	6 1-ton push cars, (4 30-ton flat cars rented)	7 push cars 3 flat cars	2 push cars 1 40-ton flat car 9 30-ton flat cars	7 push cars 9 flat cars	1 20-ton flat car
Launches....	1 26-ft. gasoline, 1 16 ft. x 70 ft. lighter	1 26-ft. gasoline		1 25-ft. gasoline	
Steel bending machines	1 McKenna type B 2 Wallace U No. 2 for $\frac{3}{4}$-in. bars 1 Wallace U No. 6 for $1\frac{1}{8}$-in. diameter 5 stirrup benders 1 elec. welding machine	2 A. Bentley & Sons power bender $1\frac{1}{2}$-in. bars 1 A. Bentley & Sons compressed air driven 6 hand benders 1 A. Bentley & Sons radius bender	2 McKenna type B for $1\frac{3}{8}$-in. bars 1 Howard Iron Works radius bender	1 McKenna type B for $1\frac{3}{8}$-in. bars 1 wire straightening machine	1 power bender designed by S. F. S. B. Co. for $1\frac{1}{4}$-in. bars
Steel shears.	1 shear (rented)	1 Toledo power shears for $1\frac{1}{2}$-in. bars	2 Shears max. bar $1\frac{3}{4}$ in.	2 Shears	2 Shears max. bars $1\frac{1}{4}$ in.
Concrete mixer	3 1-yd. Lockwood $17\frac{1}{2}$-cu. ft. Jaeger	$3\frac{3}{4}$-yd. Ransome	3 21-cu. ft. Ransome	3 24-cu. ft. Koehring	3 24-cu. ft. Koehring
Compressor .	1 350-ft. cap. with 786-ft. receiver	2 Ingersoll rand 250 ft. per min.	1 Chicago pneumatic 741 cu. ft. per min.	2 Gardiner 250 cu. ft.	1 Ingersoll rand 500 ft. per min.
Air hammers.	31 Little David	25 Ingersoll rand	30 Chicago pneumatic	30 special design	35 Ingersoll rand

The equipment of the buildings of these yards varies to a very great extent, indicating to some extent the variation in the manner in which the work is being done in the different yards.

The mold lofts at Wilmington, Jacksonville, Mobile and San Diego are all equipped with motor-driven band saws and with all small tools necessary for convenient operation.

NOT MUCH EQUIPMENT REQUIRED.

The equipment in the carpenter and joiner shops varies greatly. This variation is in part due to the amount of joiner work which the superintendent planned to do at the yard. An approximate idea of the equipment can be obtained from Table No. I. The power machines are in each case motor driven.

The equipment of the machine shops also varies greatly. Part of this equipment was installed for outfitting purposes. Table II gives an approximate list of equipment. All the power machines are motor driven.

In addition to the machinery and shop equipment noted in Tables I and II, the Jacksonville yard is equipped with four motor-driven rip saws, located on the shipways, and the San Diego yard is equipped with one rip saw and several cut-off saws located on the ways.

The variation of the construction equipment in these yards is as marked as the difference in the buildings and general arrangement of the yards. Table III below gives an approximate list of the yard equipment at each of the five yards.

The cost of the yards also varies considerably. As stated in the beginning of this article the design of the yard was left to the superintendent as much as possible in order to gain speed of construction and also to gain experience. The latest figures available indicate that the cost of the yards varies from approximately $830,000 to $1,300,000. The yards in which provision was made for outfitting naturally cost more than where no outfitting was provided for. Therefore, in each case where outfitting was provided, the cost of the yard was more than the minimum.

PROBLEMS IN THE DESIGN OF REINFORCED-CONCRETE SHIPS.

By J. Glaettli, Jr.*

This paper deals exclusively with the concrete ships designed by the Concrete Ship Section of the Emergency Fleet Corporation. It is not intended to describe the designs which were produced, since most of them have already been written up in a general way in the technical periodicals, but rather to enumerate and enlarge upon those elements which have not heretofore been published and which are of particular interest to those actively engaged in the design of concrete ships.

General Method of Design of Concrete Ships.

Feasibility.—Very little preliminary work is necessary to demonstrate the practicability of designing a concrete ship which, if constructed of ordinary concrete, will compare favorably with a wood ship of the same dimensions; and, with concrete of lighter weight than commonly used, will equal if not exceed the carrying capacity of the wood ship. The preliminary design of a concrete ship is no different than the design of a wood or steel ship in principle; but, due to the lack of experience and data, is somewhat more difficult. On account of the extra hull weight, the size of the ship for a given carrying capacity must be somewhat larger than the corresponding steel ship.

Determination of General Dimensions.—After the speed of the ship the gross displacement of the hull, the coefficient of fineness of midship section, and the block coefficient have been selected, the length, breadth and the molded depth are determined. Most of these factors are interdependent and a change in one generally necessitates a change in some of the others. The frame spacing is next determined and the approximate midship section analyzed and designed. At the same time the thickness of the shell is figured and a detailed weight estimate made. With the data now in hand, the carrying capacity of the hull can be checked and, if satisfactory, the tentative dimensions become the final ones provided the stability of the ship is within the chosen limits. Should the detailed weight fail to agree with the estimated weight used in selecting the gross displacement, then the figures are revised, using the first approximation as a guide, new dimensions are determined; and, if necessary, new designs, weight estimates and stability calculations made.

General Shape.—The determination of the length, breadth, draft and molded depth, by no means complete the preliminary design of the hull. The shape of the exterior surface must now be selected and drawn up as based on the coefficients selected. The standard method of representation

* Concrete Ship Section, Emergency Fleet Corporation, Philadelphia, Pa.

is by means of a "line drawing," which shows on one sheet the sections cut by horizontal, by vertical longitudinal and by vertical transverse planes. This line drawing forms the basis for the outline of practically all other hull and design drawings.

Systems of Framing.—It is doubtful if any marked economy of materials is effected by any particular system of arranging the interior framing of a ship, provided that designs are made on the same basis with some effort at a good arrangement. It must be conceded, however, that economy of labor is of even greater importance than a saving in materials in the first cost of a vessel. Assuming, then, that the hull weight remains unchanged for all types of framing, then that system of framing which answers all the requirements of the naval architect and the structural engineer and which involves the least labor is the one that should be adopted. The principle elements in which cost can be reduced are building of interior forms, bending and placing of main steel, and bending and placing of stirrups. The building of forms can be somewhat simplified by reducing the number of frames, introducing some longitudinals and utilizinfi those already provided. The bending and placing of steel can be reduced in cost by using straight steel in longitudinals in places of the bent steel in numerous frames. The bending and placing of stirrups can hardly be changed materially except in certain types of ships with numerous longitudinal and transverse bulkheads.

For general cargo vessels, it is probable that the best arrangement of framing is one in which frames are spaced from 12 to 16-ft. centers, with longitudinals spaced about 8 to 12 ft. apart. This arrangement permits local bending in the slab to be carried transversely to the ship, and relieves the principal steel in the deck and bottom of this stress. It requires the use of thick slabs, which will better resist local impact and is an easier structure to build. For vessels with numerous transverse bulkheads, it is possible to do away with frames entirely, using a system of longitudinals supported on the bulkheads.

The relative merits of the two systems of framing for cargo ships, namely, frames only and frames plus longitudinals, have been frequently discussed and the general conclusions may be summed up as follows: That as far as economy of materials is concerned, only a very refined calculation, not warranted by the accuracy of the assumptions made, will show any difference between the two systems. That economy in cost as determined from the labor required in construction will be the governing factors in deciding which system should be used.

STRENGTH CALCULATIONS.

Longitudinal Strength.—The method of making the calculations for the moments and shears in a vessel among waves due to hogging and sagging, is explained in text-books on naval architecture* and will not be repeated

* See "Strength of Ships," by A. J. Murray. "Naval Architecture," by O. H. Peabody. "Text Book of Theoretical Naval Architecture," by E. L. Attwood.

here. For purposes of design and comparison, the longitudinal strength calculations were made for all designs. These calculations are rigid for the assumptions made, and hence are apt to convey a false impression of their absolute value. Their principal value lies in the fact that they serve as a means for comparing the longitudinal strengths of different ships, and, in lieu of more accurate information, they constitute a basis for design.

In view of the uncertainty of hydrostatic pressures in the trough and the crest of a wave, the uncertainty of the relation of wave height to length, the possibility of unfavorable loading of cargo, and the effect of the upward and downward acceleration of a ship among the waves on the weight of the vessel, the results do not accurately represent the actual extreme conditions which may be met. These variable elements may each of them be small, but the totals may be cumulative, and may materially increase or diminish the results obtained by calculations. The value of an elaborate series of tests extending over a long period, to determine the actual longitudinal stresses cannot be overestimated. Reliable information regarding the longitudinal strength required will contribute more to the production of economical designs than years of experience. If too much steel is being placed in the hulls of concrete to resist longitudinal bending, experience alone will not demonstrate the fact. This longitudinal steel represents between 40 and 50 per cent of the total steel in a concrete ship.

Transverse Stress Calculations.—The work of designing the structural members involves the calculation of moments and shears in the transverse frames of the ship, which, being closed frames, require a solution by the methods used in solving statically indeterminate structures.

The general character of concrete ship construction and the possibility of impact and of reversal of stresses makes it necessary to provide tensile reinforcement near more than one face of the members. Where two or more members meet, as at the joints of a frame, the steel is usually carried into the other member, or members, in such a manner as to allow stresses to be transmitted from one to the other through the joint. The rigidity of the joints may cause stresses in adjoining members nearly as great as in the member to which the loads are applied; in some cases, the loads on one member can cause a reversal of stress in others. Consequently, a concrete ship frame should be treated as a unit, and a careful investigation made of the distribution of bending moment and shear for the various conditions of loading to be expected in service. A very satisfactory method of determining moments and shears in a closed frame was developed by G. A. Maney.*

In the application of the moments and shears to the design of members, particular attention should be paid to the slab steel acting with the frame members.

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* Bulletin I, University of Minnesota, "Secondary Stresses and other Problems in Rigid Frames," March, 1915; Bulletin 80, University of Illinois, June, 1915.

DETAIL DESIGN OF CONCRETE SHIPS.

Naval Architecture as Applied to Concrete Ships.—The term "vessel design," as used in connection with reinforced-concrete ships, is understood to cover the outlining of the general type of vessels to meet the conditions imposed, the fixing of dimensions, freeboard and displacement, the selection of coefficients of form, the estimating of power, weight and stability, and calculations relative to the forces acting on the ship girder when the vessel is in various conditions of loading among waves.

Judgment relative to the naval architectural features of concrete vessels should take cognizance of the fact that the material under consideration is essentially different from the materials heretofore used in the construction of ships, and that men experienced in reinforced-concrete work are not in general familiar with ships and the viewpoint of ship men. It has been the policy of those responsible for the naval architecture of concrete vessels to conform, so far as practicable, to the ideas of concrete engineers in order that the material might be utilized under the most advantageous circumstances possible.

Compliance with the principles just outlined has not resulted in the slighting of important naval architectural considerations, but it has somewhat modified the principal characteristics of certain of the vessels designed, so that they do not conform exactly to practice in steel and wood ship building. The changes which have been made, however, are not believed to have affected the design deleteriously. With the coöperation which has been obtained from the representatives of the classification societies, as well as other men of experience in ship construction and ship handling, it is believed that designs have been produced, which, as a whole, embody good ship building practice, even when viewed from the standpoint of men not essentially interested in the success of concrete ships.

Problem of Structural Design.—While the solution of the problems involved in the design of the concrete members has been no easy matter, the number of problems involved has been comparatively small, and experience alone will demonstrate whether they have been correctly solved. Successive designs have in general shown a decided improvement over those that have gone before and unquestionably the latest designs are far better than the earlier ones. The individual problems that arose are discussed in detail in the following paragraphs.

Continuous Beams and Compression Reinforcement.—In order to reduce the weight of beams, frames and slabs to a minimum, all members, whose cross-section was largely determined by the intensity of the moment to be resisted, were designed as continuous beams wherever the construction was such as to enable this condition to be realized. In many instances, compressive reinforcement was added in order to reduce the gross cross-section of members, but in the usual case reinforcement was used on both faces to provide for reversing moments or to provide reinforcement for negative moment over the supports.

Percentage of Steel.—At the very beginning of the work of design, it became evident that larger percentages of steel would be required in the design of concrete members than had heretofore been used in common practice. It also became evident that the resistance to shear or diagonal tension would have to be increased by using more steel or by using it in a different arrangement. Higher unit stresses in the concrete have also been found desirable to a limited extent. The use of the working stresses adopted has been fully justified by the experimental work done.

The amount of steel used in the various designs ranges from 3 to 4 per cent of tensile steel and from 1 to 4 per cent of compressive steel. If web reinforcement be included, the total amount of reinforcing steel in concrete ships thus far designed by the Concrete Ship Section ranges from 5 to 9 per cent by volume, or from 22 to 40 per cent by weight, assuming concrete weighing 110 lb. per cu. ft.

Unit Stresses in Flexure.—There seems to be a well-established opinion among those not familiar with concrete ship design that the unit stresses in a concrete ship are materially greater than in those used in land structures. This, however, is not the case, and it can be said that, with few exceptions, the stresses are as conservative, if not more so, than the stresses used in land structures. In a great many cases members are subjected to reversing moments, a condition which requires steel on both faces of the member. If these moments are equal in magnitude, the amounts of steel on each face must be nearly equal, and the compressive strength of the concrete is then of no exceptional advantage, the tensile steel on one face being sufficient to carry the compressive stresses when the moment is reversed. However, to keep the structure intact, a safe working stress of from 1000 to 1500 lb. per sq. in. must be permitted in the concrete. In view of the fact that in certain parts of a ship the steel stresses must be kept low to prevent leakage, we find numerous instances where the stresses are more conservative than usually supposed. In very few instances have the unit stresses in the steel for ordinary service conditions been increased above 16,000 lb. per sq. in.

Unit Stresses in Shear.—The usual method of designing shear members on land structures has been to assume that a large proportion, say 75 per cent of the shear, is carried by steel stirrups at a unit stress of about 75 per cent of the usual tensile working stress in the steel. In the design of concrete ship members, except the shell, it has been the practice to assume that all of the shear is carried by steel stirrups at 16,000 lb. per sq. in.; this method giving the same results as obtained in common practice. It has been assumed that the webs of beams could be reinforced to the extent of permitting a maximum working shearing unit stress of 500 lb. per sq. in., and then assuming that all of the shear or diagonal tension is carried by stirrups. For this item alone can it be said that radical departures have been made from common practice. The results of tests justify the use of such large percentages of web steel.

Reinforcing the Shell for Shear.—The side shell of a ship is subjected

to shearing stresses of a reversing character and the best arrangement of steel to resist such shearing stresses is not easily determined. It is generally conceded that diagonal rods are more effective than vertical rods for resisting diagonal tension, especially before the concrete has cracked. If the shears are reversible two sets of diagonals must be provided at right angles to each other, while only a single set of vertical rods is needed.

Using the same unit tensile stresses in the vertical as in the diagonal bars, as was done in the first ship designed by the Concrete Ship Section, the amount of vertical steel required is only half as great as the amount of diagonal steel needed. However, with the use of vertical rods to resist shear, it becomes necessary to add horizontal rods to tie the structure together, and the apparent advantage of the vertical steel is somewhat reduced. In addition, the disadvantage of having horizontal rods in thin vertical slabs at the time of pouring must be considered.

It can be demonstrated analytically that vertical shear reinforcement takes little or no stress as long as the concrete remains intact and laboratory experiments bear this out. It is to be expected then that diagonal reinforcement is the more effective in preventing cracks from forming and in keeping the width of the cracks down once they do form. The work of the investigation branch of the Concrete Ship Section indicates that lower working stresses should be used in vertical rods than in diagonal rods in order to keep the size of cracks down to a minimum.

When using a materially reduced working stress for vertical bars, in the design of the side shell of a ship, it is found that the amount of vertical steel required is about 90 per cent of the amount of the total steel in both sets of diagonals. The use of horizontal rods in conjunction with the vertical rods eliminates even this saving of 10 per cent, and less steel is required if placed in a diagonal direction.

It does not follow necessarily that diagonal steel reinforcement should be used in preference to vertical. The problem is invariably complicated by the conditions of local bending that must be provided for. Diagonal steel is generally not as effective for carrying local bending as are rods which span the panel in the direction of the shortest dimension. It is probable that a combination of vertical and diagonal steel is as efficient and economical as can be expected. What proportion of shell steel should be placed vertically and what amount diagonally depends on the thickness of the slab, the direction in which local bending is carried, the sizes of rods to be used in the shell, the maximum and minimum spacing of rods and other features.

Kind and Size of Rods Specified.—Nearly one-half of the steel in a concrete vessel is in the deck and bottom, where the bond stress is exceedingly low, and where deformed bars have no advantage over plain rods. This feature and the difficulties which have arisen in the field with deformed rods have led the Concrete Ship Section to specify plain round rods for all vessels.

While large diameter rods have been used in certain designs with a

view to reducing the number of pieces to be handled, the difficulties encountered in bending the steel have demonstrated that the rods should be as small as can be conveniently placed in beams and slabs. In slab design particularly should small rods be used. The use of small steel rods (less than ¾ in. diameter) in the shell is especially encouraged because their use eliminates all previous bending of the steel for the turn of the bilge. The size of rod best suited for a particular location depends on the thickness of the member, the number of bends, etc., as well as on the requirements of design.

Thickness of the Cover Over Rods.—A simple calculation will show that if the upper deck and shell of a 7500-ton ship can be reduced in thickness ⅛ in., there will result a net saving of roughly 30 tons in the hull weight and a corresponding increase in dead weight carrying capacity. It is therefore of prime importance that the cover of slab steel be reduced to the minimum required or the protection of the steel. Foreign practice seems to consider 1 centimeter sufficient, a figure which compares favorably with the ⅜-in. cover used by the Concrete Ship Section. The above figures can be applied to illustrate how important it is to build forms exactly to size and to hold them rigidly in place. The inner and outer bottoms, decks and other slabs built without forms should also provide no more than the cover called for on the plans.

HULL EQUIPMENT OF CONCRETE SHIPS.

In connection with concrete ships, the design of hull details is of interest principally from the standpoint of the methods of attaching equipment to the concrete, and this phase of the work is taken up to the exclusion of other features which are common to steel, concrete and wood ships.

Standard Equipment Used.—The equipment of a concrete ship need differ but little from that of a steel or wood ship. It is probable that as concrete ship design becomes more common, that equipment will be devised especially suited for easy attachment to concrete decks; but, up to the present time, and especially during the war, when every effort was directed to standardizing equipment, no effort has been made in such directions. The regular equipment of steel ships was used with as little modification as possible and suitable connections designed. Experience only will tell whether the details will prove satisfactory. Should specially designed equipment be produced for concrete ships, it should in general have a larger base whenever it is attached to the concrete, in order to distribute the pressures over large areas.

Anchor Bolts.—Considerable difference of opinion exists as to the proper method of handling anchor bolts. The drilling of the concrete for bolted connections is undesirable, if not altogether impossible, on account of the large amount of steel reinforcement. The setting of anchor bolts in the concrete at the time of pouring has its drawbacks in that there are from three to four thousand bolts to be set in one of the larger vessels.

Using through bolts and placing thin metal sleeves in the forms large enough to permit flushing grout between the bolt and the sleeve after the equipment is set is one method which will prove satisfactory and which has been largely used. The advantages of this method are that since sleeves are over size, some leeway is allowed in setting them, while the grouted through bolt is as secure as an ordinary anchor bolt. The disadvantages are that the sleeves are almost as easily displaced as anchor bolts. The final solution will probably be: *First*, that small fittings will be provided and placed before the concrete is poured, using the base as a template; *second*, that for the larger fittings, a template, preferably of metal, will be made to rigidly hold the necessary anchor bolts in position; this template to be set in place in the forms and to remain a part of the finished structure; *third*, in the case of certain equipment, the manufacturer will provide a base specially designed to be embedded in the concrete, and to rake the fitting by a simple bolted connection. These recommendations are made with the conviction that field drilling is an unnecessary and expensive operation inherited from the steel ship, and is almost entirely dispensable in a concrete vessel.

Location of Inserts.—The most important work, so far as equipment is concerned, is the location of inserts. Unfortunately, it is not as easy to add openings or relocate holes in concrete as in steel, and, almost without exception, anchor bolts, inserts of various kinds, the sterm fram, mooring rings, hawse pipes, chain pipes, rudder trunk and the like, must be provided and definitely located before the concrete is poured. This rule is not inflexible, as certain equipment can be grouted into openings left in the concrete, but, in general, to produce a water-tight job, it should be cast in place.

It follows, therefore, that all design work, both structural and hull detail, must be essentially completed, up to and including the upper deck, before construction can proceed very far. This is entirely different from the case of a steel vessel, where the construction may begin as soon as the keel plan is finished, the other plans being developed as needed. The rapid production of plans and, consequently, a large drafting force, is, therefore, a peculiar necessity in concrete ship design.

Stern Frame.—Up to the present time no special effort has been made to devise a stern frame detail and arrangement particularly suited to the concrete ship. Instead, the standard steel stern frame for single screw steel ships has been adopted and attached to the concrete by suitable anchors. It would seem to be entirely feasible to do away with the major portion of this stern frame, by using a special casting to hold the end of the stern tube, and by using a carefully designed balanced rudder. This has been done on steel ships, and should prove possible on concrete ships. If the standard type of stern frame is to be used the details should be changed so as to provide a simple method of attachment to the concrete.

Stem Protection.—While a stem plate has not been provided on all our ships, experience will probably show that it is not only desirable but

necessary in order to prevent the stem from being scarred up when fouling the anchor chain.

Experience will in all probability also show that in the way of hawse pipe, the bow, from the light water line to the hawse pipe, should be protected from the flukes of the anchor.

MACHINERY INSTALLATION.

Similarity to Steel Ship Practice.—As would be expected, the installation of propelling machinery follows steel and wood ship practice very closely. No fundamental principles are changed and little opportunity is afforded to make use of the special properties of reinforced concrete. With the exception, then, of the details where piping, line shaft, etc., pass through bulkheads, and where sea connections are made, little difference can be found in the arrangement of machinery between concrete and other vessels. For obvious reasons no great difference can exist, and such improvements as might have been made have been neglected on account of lack of time.

One of the important problems in connection with machinery installation was the design of standard inserts which could be embedded in the bulkheads, shell and deck, to be used for passing piping through watertight slabs and for making connections to the outside.

Suggested Changes from Steel Ship Practice.—The opportunities of using reinforced concrete to advantage in the engine and boiler room are limited, but the few that do exist should not be neglected. It is entirely feasible to omit all steel girder foundations under the engines and boilers, using reinforced-concrete beams instead. Engines and boilers can be aligned and the foundation plates grouted to a uniform bearing, as in standard stationary engine practice. Marine men, however, do not strongly favor such construction, and in most cases a certain amount of structural steel has been retained in the engine and boiler foundations. Experience will most likely demonstrate that all of the structural steel may be eliminated in the boiler room. The engine foundation will have to be of such type, that the engine may be realigned should the vessel acquire a permanent hog or sag.

The possibility that the holding down bolts may have to be replaced must not be overlooked and any design for foundations for engines should be arranged in such a way that these bolts may be renewed without too much difficulty and expense.

One opportunity of using reinforced concrete to advantage in the engine and boiler rooms has so far been neglected. The waste spaces between frames and bulkhead stiffeners are admirably adapted for use as tanks, which can be more economically built of concrete than of steel. Such an arrangement would economize on space in the engine and boiler room, give a double side wall as well as double bottom, with a consequent added strength and security, and would answer the requirements of most of the tanks.

CONCLUSION.

It should not be inferred from the foregoing that the design of the general feature of concrete ships constitutes the major proportion of the work.

When it is considered that a ship is the largest movable structure built by man, that it is fully equipped to be the permanent home of a numerous crew, that it contains an exceptionally complete power plant in a very restricted space, and is equipped with much other necessary boat and cargo handling machinery, it is instantly realized that an enormous amount of detail design work, much of it new and without precedent, must be accomplished before construction can be begun.

In the future development of the concrete ship a great deal of attention must necessarily be directed toward the simplification and standardization of these details.

METHOD OF CONSTRUCTION OF CONCRETE SHIPS.

By R. J. Wig.*

This paper is confined to a description of the methods employed in the construction of concrete ships for the United States Shipping Board Emergency Fleet Corporation.

The complete program of the Fleet Corporation comprises the construction of 14 ships of five different types. The characteristics of these ships are given in Table 1. Two of the above ships were considered of experimental character and constructed under contract in private yards. One of 3000 tons D. W. was designed by the Liberty Shipbuilding Co., Wilmington, N. C.

Table I.—Characteristic Data on Concrete Ships.

Type of Ship Design Number or Name	Cargo. Atlantus	Cargo. No. 2.	Cargo. Polias.	Cargo. No. 69.	Tanker. No. 70.
Length, B.P.	250–0	268–0	268–0	420–0	420–0
Length, O.A.	260–2½	282–2	281–9	434–3	434–3
Beam (max.)	43–6	46–0	46–0	54–0	54–0
Draft, full cargo	22–6	23–6	23–6	26–0	26–0
Depth	26–9	28–3	26–6	36–0	36–0
Displacement, full cargo	5,225	6,175	6,220	13,000	13,000
Deadweight	2,620	3,257	3,274	7,499	7,445
Speed in knots	10½	10½	10½	10½	10½
Metacentric height (cargo—light)	:	2.18:1.87	:	2.27:3.20	2.60:3.30
Frame spacing	5′–0″	5′–0″	4′–0″	12′–9″	4′–3″
Shell thickness—bottom	5″	5″	5″	4¾	5
" " —side	5″ and 5½″	4″	5″	6½	4
Type of deck erections—poop	Concrete	Wood	Wood	Wood	Wood
" " " " —bridge	Concrete	Wood	Wood	Concrete	Concrete
" " " " —forecastle	Concrete	Wood	Wood	Concrete	Concrete
Reinforcing bars—weight in tons	446	452	....	1,375	1,370
Concrete, volume in cu. yd.	1,092	1,120	1,100	2,550	2,660
Type engine	Triple exp.	Triple exp.	Triple exp.	Triple exp.	Triple exp.
Developed Ihp.	1,520	1,520	1,520	2,800	2,800
Developed RPM	94	94	94	....	....
Boiler	Water tube	Water tube	Water tube	Water tube	Water tube
Fuel	Oil	Oil	Coal	Oil	Oil

The hull was built by them in a private yard at Brunswick, Ga. The outfitting and installation of machinery was done under contract by the American Shipbuilding Company of Brunswick, Ga. The other chip was of 3500 tons D. W. and was designed by the Fougner Concrete Shipbuilding Co. of New York. The hull was constructed by the designers at Flushing Bay, N. Y., but the outfitting and installation of machinery was done by the Lord Construction Company of Providence, R. I.

The remaining 12 ships were constructed in yards† owned by the Emergency Fleet Corporation and especially designed and built for the purpose.

* Head, Concrete Ship Section, Emergency Fleet Corporation.

† For description of yards see paper by A. L. Bush "Layout and Equipment of the Government Concrete Shipyards." Pro. Am. Concrete Institute, 1919.

The yards were built and operated by contractors who acted as agents for the corporation. Following is a list of the contractors and a statement of the location of the yards and the number and type of ships built:

The San Francisco Shipbuilding Co. operating the yard at Oakland, Calif., are constructing complete two 7500-ton D. W. concrete oil tankers (Type 70), Fig. (1) a and b, and one 7500-ton D. W. cargo ship (Type 69), Fig. (2) a and b. The Pacific Marine and Construction Co. are operating the yard at San Diego, Calif. They are constructing complete two 7500-ton D. W. concrete oil tankers (Type 70). The Fred T. Ley & Co., Inc., of Boston, Mass., are operating the yard at Mobile, Ala., and are constructing complete two 7500-ton D. W. concrete oil tankers (Type 70) and one 7500-ton D. W. cargo ship (Type 69). The A. Bentley & Sons Co., of Toledo, Ohio, are operating the yard at Jacksonville, Fla., and are constructing the concrete hulls for two 7500-ton D. W. oil tankers (Type 70). These tankers are being outfitted by the Jacksonville Ship Outfitting Yard at Jacksonville, Fla. The Liberty Shipbuilding Co. are operating the yard at Wilmington, N. C., and are constructing two 3500-ton D. W. concrete hulls (Type 2), Fig. 3. The outfitting is being done by the Jacksonville Ship Outfitting Co.

The reinforced-concrete ship is an ultra-refined concrete structure with walls, floors and columns similar to a building, requiring the engineer to deal with forms, reinforcement and concrete, but with such refinement in quality and workmanship that new methods of construction had to be developed or old ones improved in order to meet the exacting conditions. Forms on a large scale had to be built of great rigidity to conform to irregular curves and shapes and exacting dimensions, and supported by novel means. Reinforcing steel of both large and small diameter and long and short lengths had to be bent with exactness to irregular curves and firmly secured in position with very small tolerances on account of limited space made necessary by the requirement for minimum weight of hull and small allowable covering of concrete. Concrete of excellent quality but the lightest possible weight had to be placed in thin walls containing very high percentages of reinforcing steel, in many cases the bars being only a few diameters apart.

Unlike a building the concrete ship must be constructed on a temporary foundation or underpinning.

The Emergency Fleet Corporation adopted the policy of suggesting feasible methods to the contractors but permitting each to develop and use its own methods, provided the cost was not excessive. Thus the methods employed for the various operations differed in the several yards.

Complete cost records have been made of each operation and comparisons can be made of the relative efficiency of the several methods.

Before discussing in detail the methods employed by the contractors for the various operations it would perhaps first be well to outline briefly the stages through which the construction work progresses, giving the reader a general bird's-eye view of the entire operation. The order of procedure is not fixed, for many of the operations would overlap and it was not possible at times to proceed with the work as desired because of lack of certain required materials, but the outline given below was essentially followed:

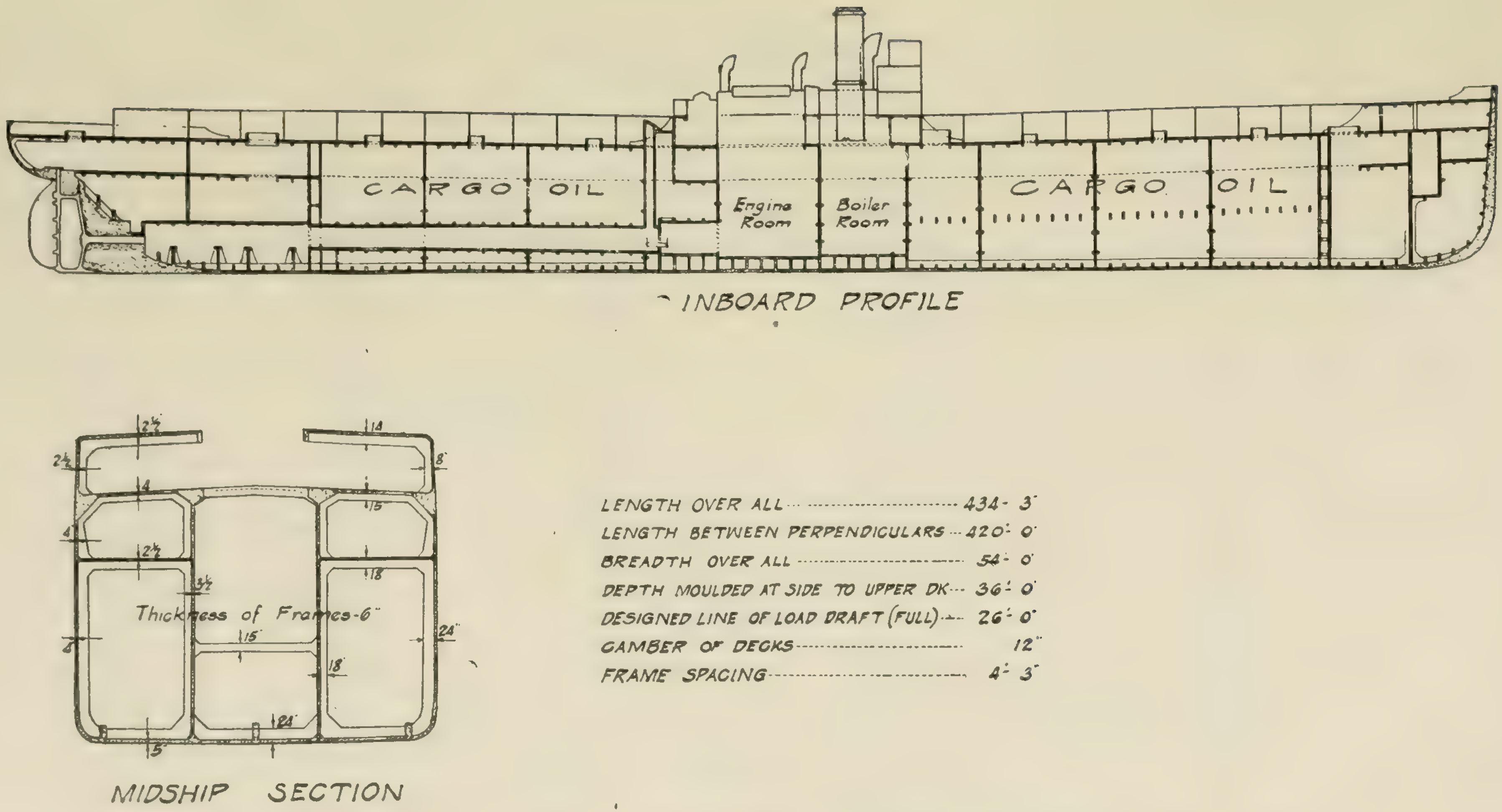
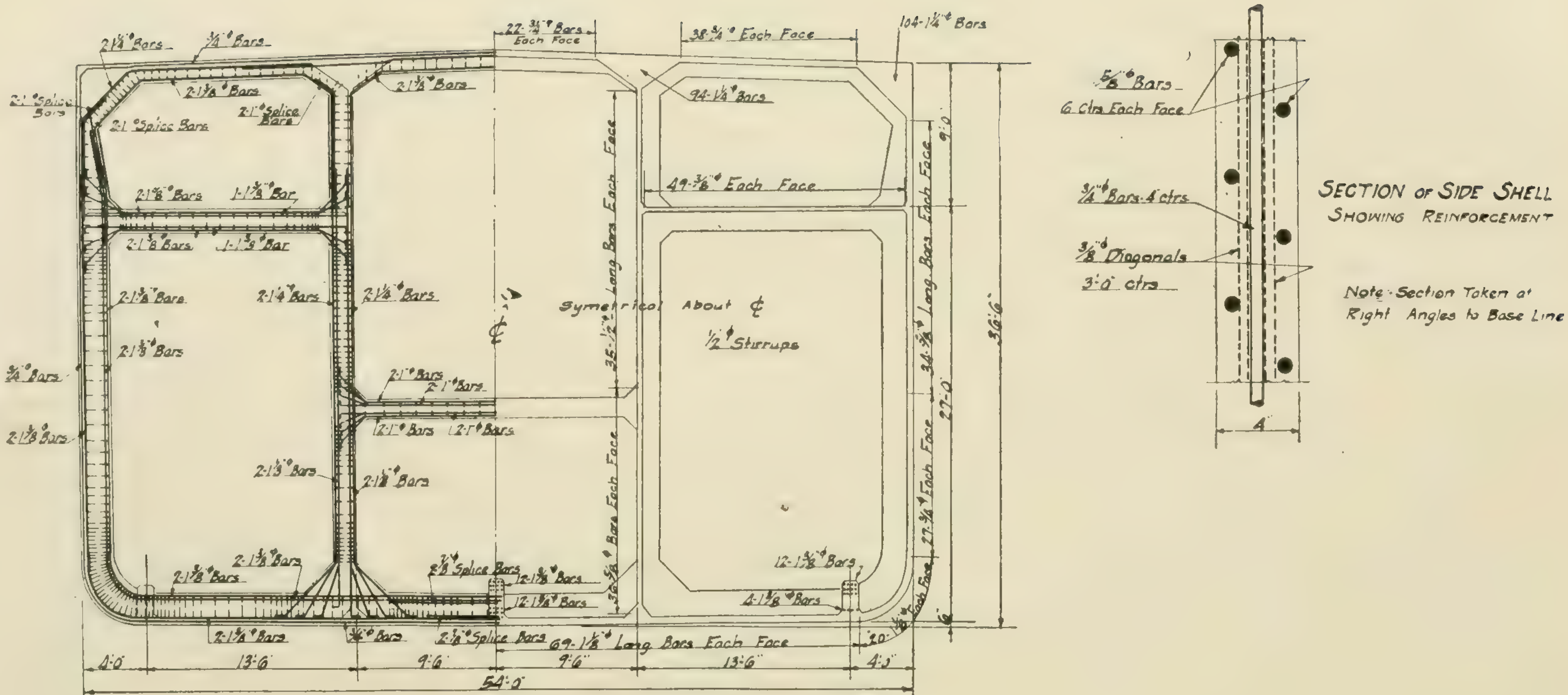


FIG. 1-A.—INBOARD PROFILE AND MIDSHIP SECTION 7,500 D.W.T. REINFORCED CONCRETE TANKER TYPE NO. 70.



TYPICAL MIDSHIP SECTION
Showing Frame Steel

FIG. 1-B. —TYPICAL MIDSHIP SECTION, SHOWING STEEL, TYPE NO. 70.

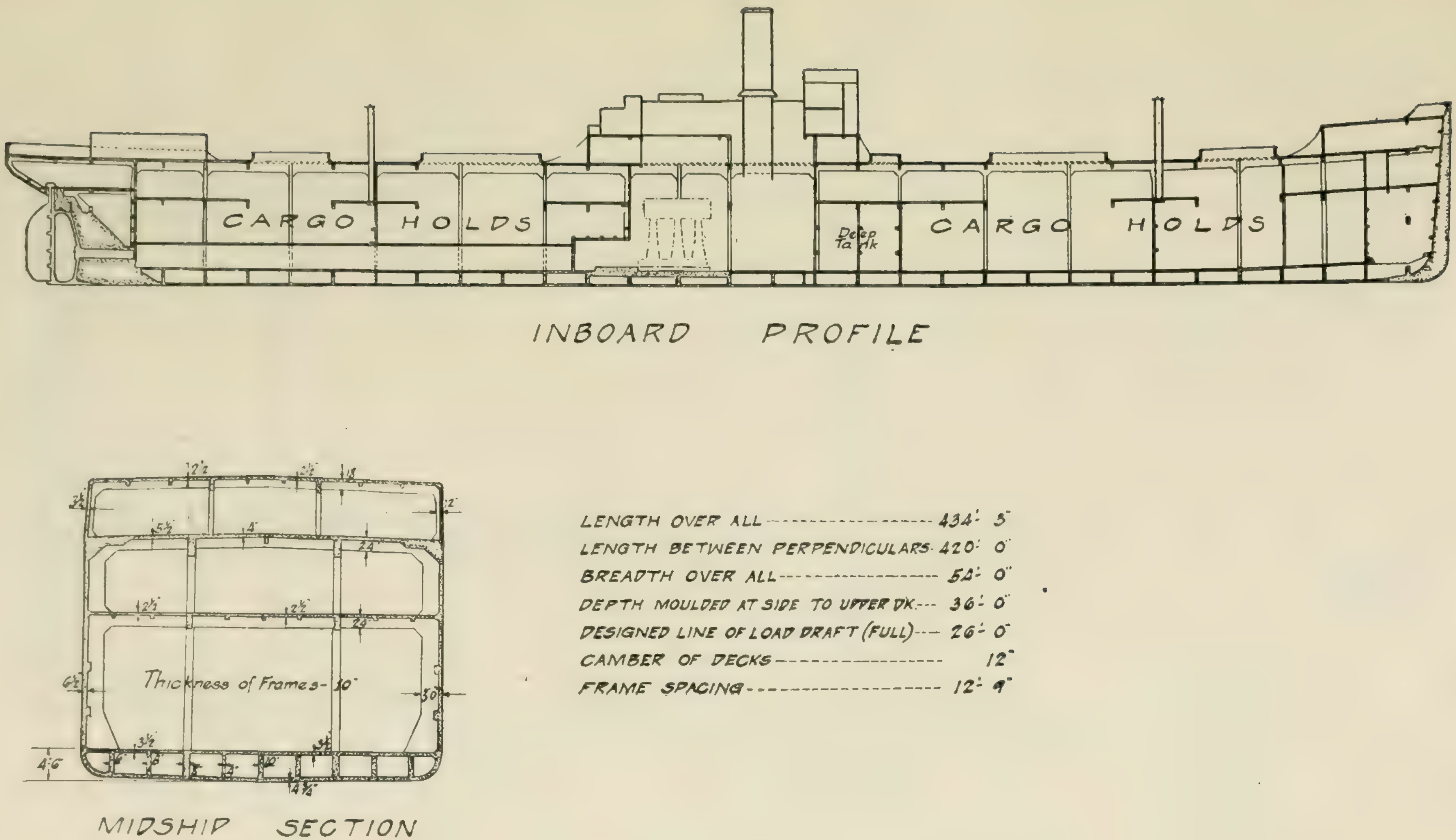


FIG. 2-A.—INBOARD PROFILE MIDSHIP SECTION 7,500 D.W.T. CONCRETE CARGO SHIP TYPE NO. 69.

1. The underpinning or blocking for supporting the floor forms is set in position on the ways. (Fig. 4.)
2. The scaffolding with overhead trusses for holding the outside forms is set in position. (Figs. 4 and 5.)
3. The outside bottom and side forms are erected complete thus providing means for supporting the reinforcing steel. (Fig. 5.)
4. All steel inserts such as sea chests, stern frame, stem plate, hawse pipes, etc., are secured in place on the inside of the outside forms. (Figs. 5, 6, 7, 12 and 32.)
5. The bottom and side shell reinforcing steel is placed within the outside forms. (Fig. 7.)
6. The bottom and side frame steel and the keelson reinforcing steel is erected. Splice bars between the bulkheads and shell are placed in position. (Fig. 8.)
7. The inside frame, keelson and side shell forms are erected to the height of four or five feet. (Fig. 9.)
8. Concrete is placed in the keelsons and in the bottom and sides of shell and frames up to the 4 or 5-foot draft line. (Fig. 10.)
9. Bottom inside forms are removed and the concrete is pointed up where necessary. Top surface of concrete where it will join the succeeding pour of concrete is thoroughly cleaned and roughened. (Fig. 11.)
10. The erection of the frame and bulkhead steel is continued up to the elevation of the second deck. (Fig. 12.)
11. The inside frame, bulkhead and shell forms are erected up to the second deck. (Fig. 13.)
12. Inserts for pipes, equipment, etc., in frames and bulkheads are set in place. (Fig. 11.)
13. Concrete is placed up to the underside of the second deck. (Fig. 14.)
14. The inside forms are removed from the frames and bulkheads, the concrete is pointed and the upper surface which makes a joint with the next lift is cleaned and roughened.
15. Forms for the second deck beams and slab are placed supported on inside staging and props from the concrete keelsons and frames. (Fig. 15.)
16. Inserts for equipment and pipes in the second deck are placed.
17. The reinforcing steel in the second deck is placed. (Fig. 16.)
18. The concrete is placed in the slab and beams of the second deck. (Fig. 17.)
19. The inside frame and bulkhead reinforcing steel is erected to the top deck.
20. The inside shell, bulkhead and frame forms are erected to the top deck. (Fig. 18.)
21. Inserts for equipment and pipes in frames and bulkheads between the second and top decks are placed.

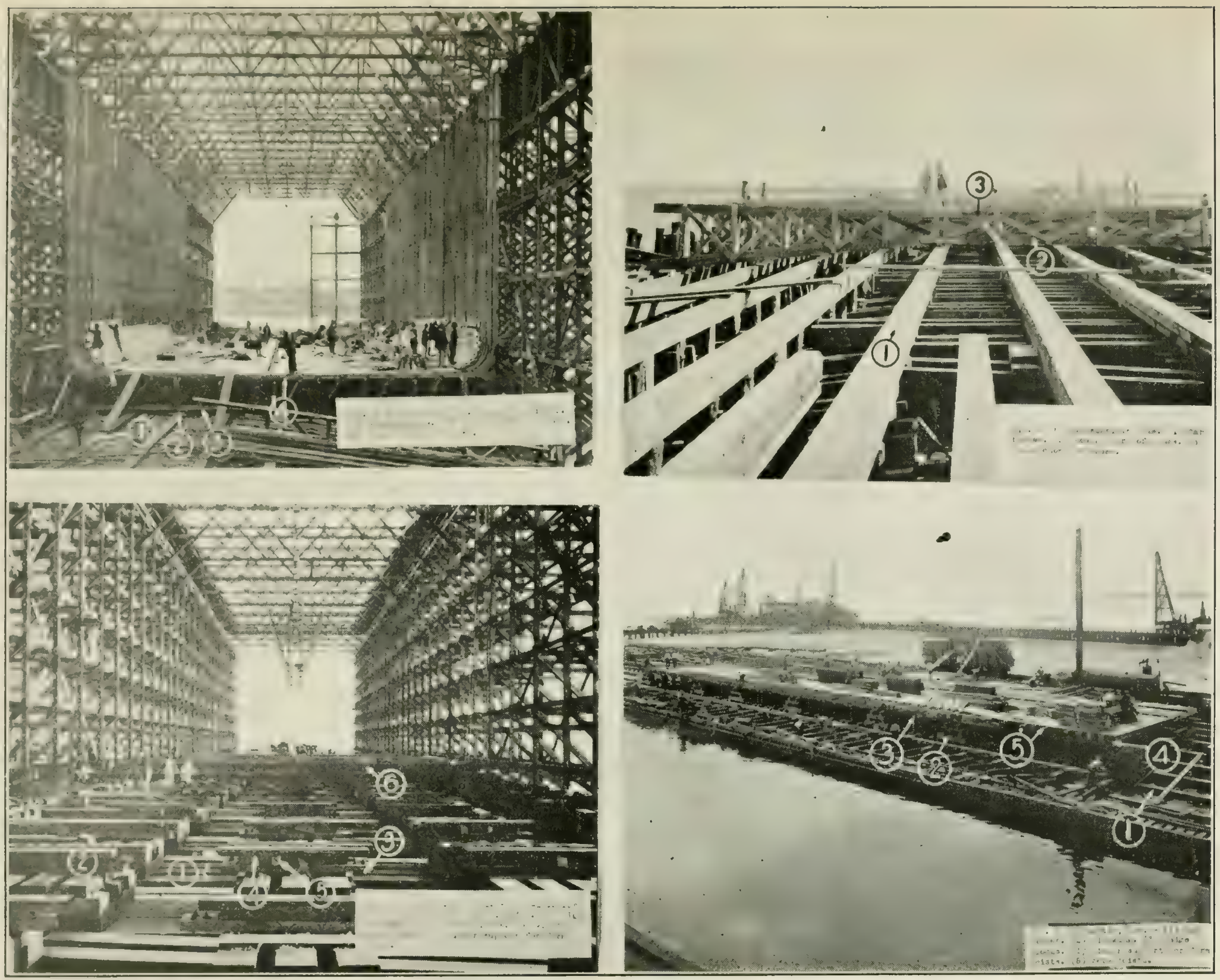


FIG. 4.—UNDERPINNING OR BLOCKING FOR SUPPORTING THE FLOOR FORMS, AND THE SCAFFOLDING, WITH OVERHEAD TRUSSES FOR SUPPORTING THE OUTSIDE FORMS, ARE ERECTED.

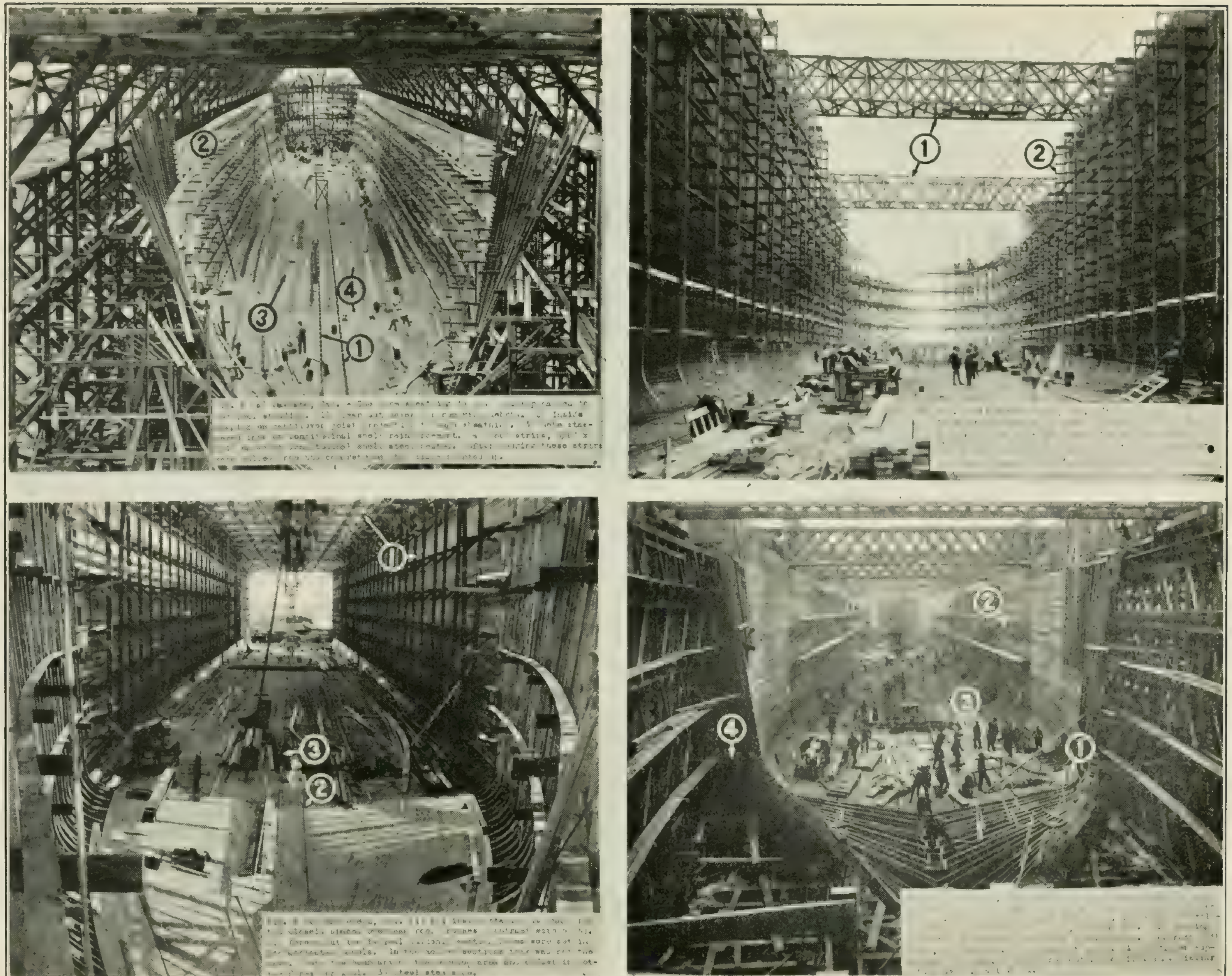


FIG. 5.—OUTSIDE BOTTOM AND SIDE FORMS ARE ERECTED COMPLETE.

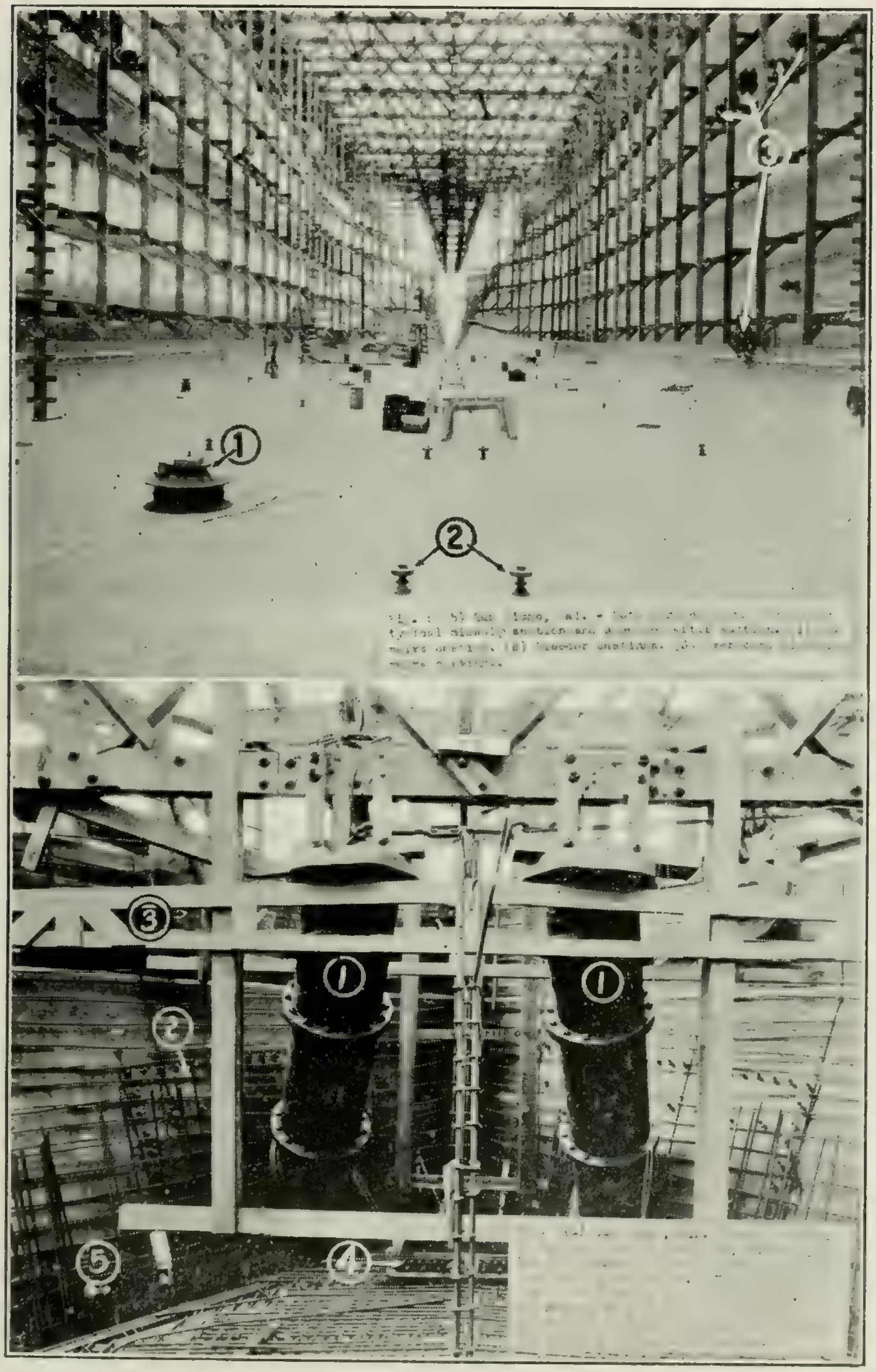


FIG. 6.—DETAILS OF INSERTS. ALL STEEL INSERTS SUCH AS SEA CHESTS, STERN FRAME, STEM PLATE, ETC., ARE SECURED IN PLACE ON THE INSIDE OF THE OUTSIDE FORMS. THE SEA CHESTS AND OTHER OUTBOARD INSERTS WERE SIMILAR FOR ALL SHIPS. THE DESIGN OF THE ATTACHMENT OF STEEL STERN FRAME TO CONCRETE HULL WAS DIFFERENT IN ALL YARDS. (SEE FIG. 29.)

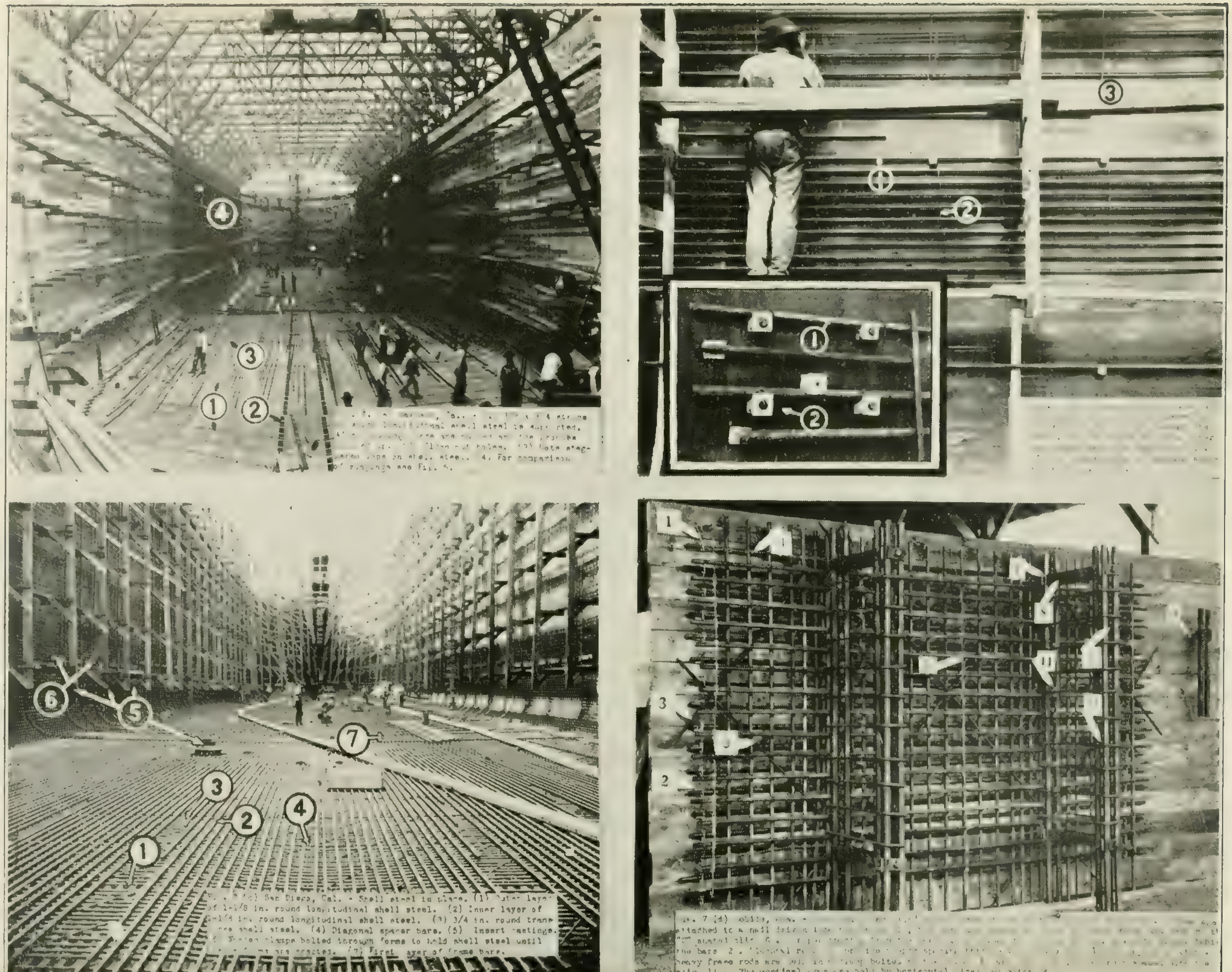


FIG. 7.—BOTTOM AND SIDE SHELL REINFORCING STEEL PLACED WITHIN THE OUTSIDE FORMS.

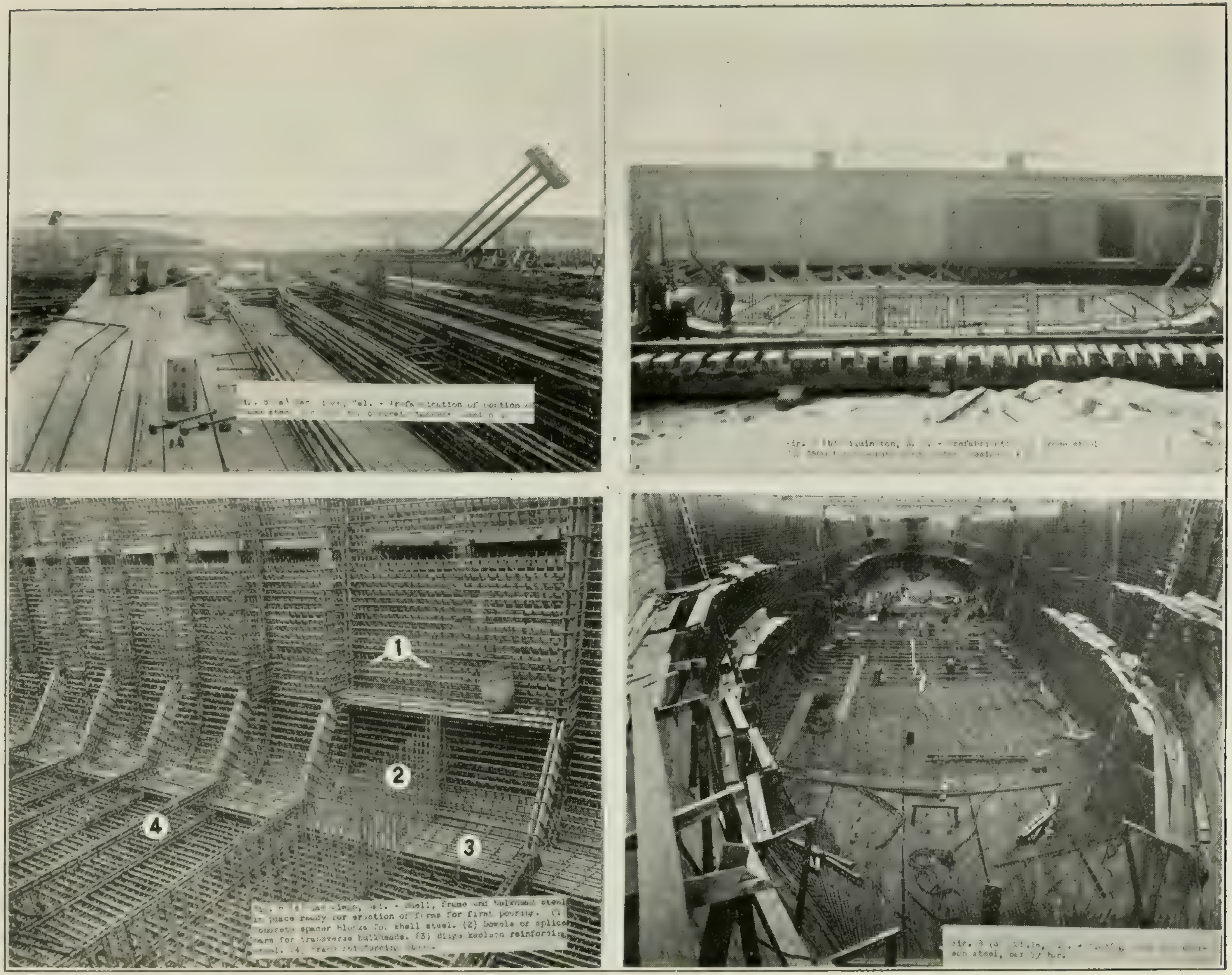


FIG. 8. BOTTOM AND SIDE FRAME STEEL AND THE KEELSON REINFORCINGSTEEL IS ERECTED. SPLICE BARS BETWEEN THE BULKHEADS AND SHELL ARE PLACED IN POSITION.

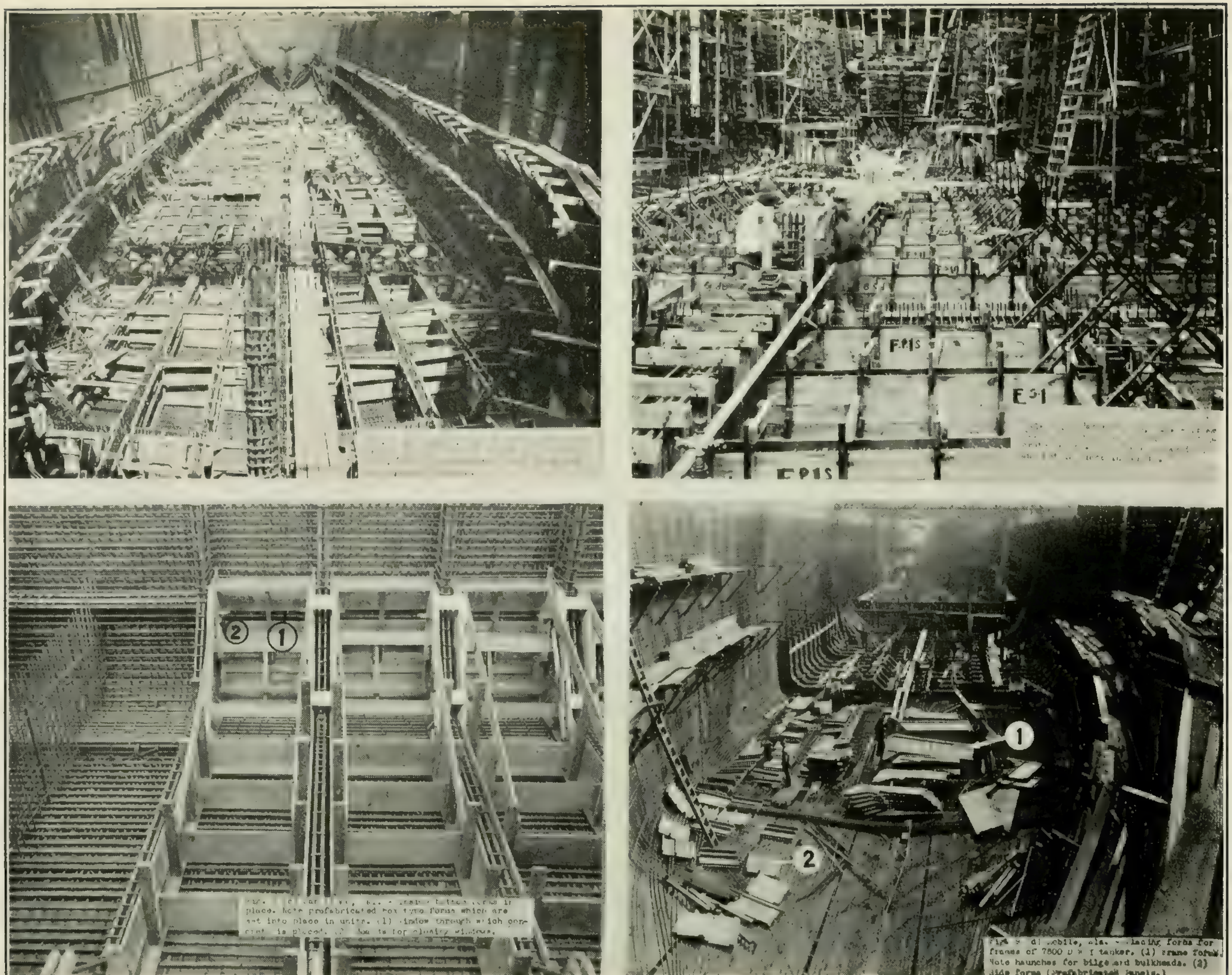


FIG. 9.—INSIDE FRAME KEELSON AND SIDE SHELL FORMS ERECTED TO THE HEIGHT OF FOUR OR FIVE FEET.

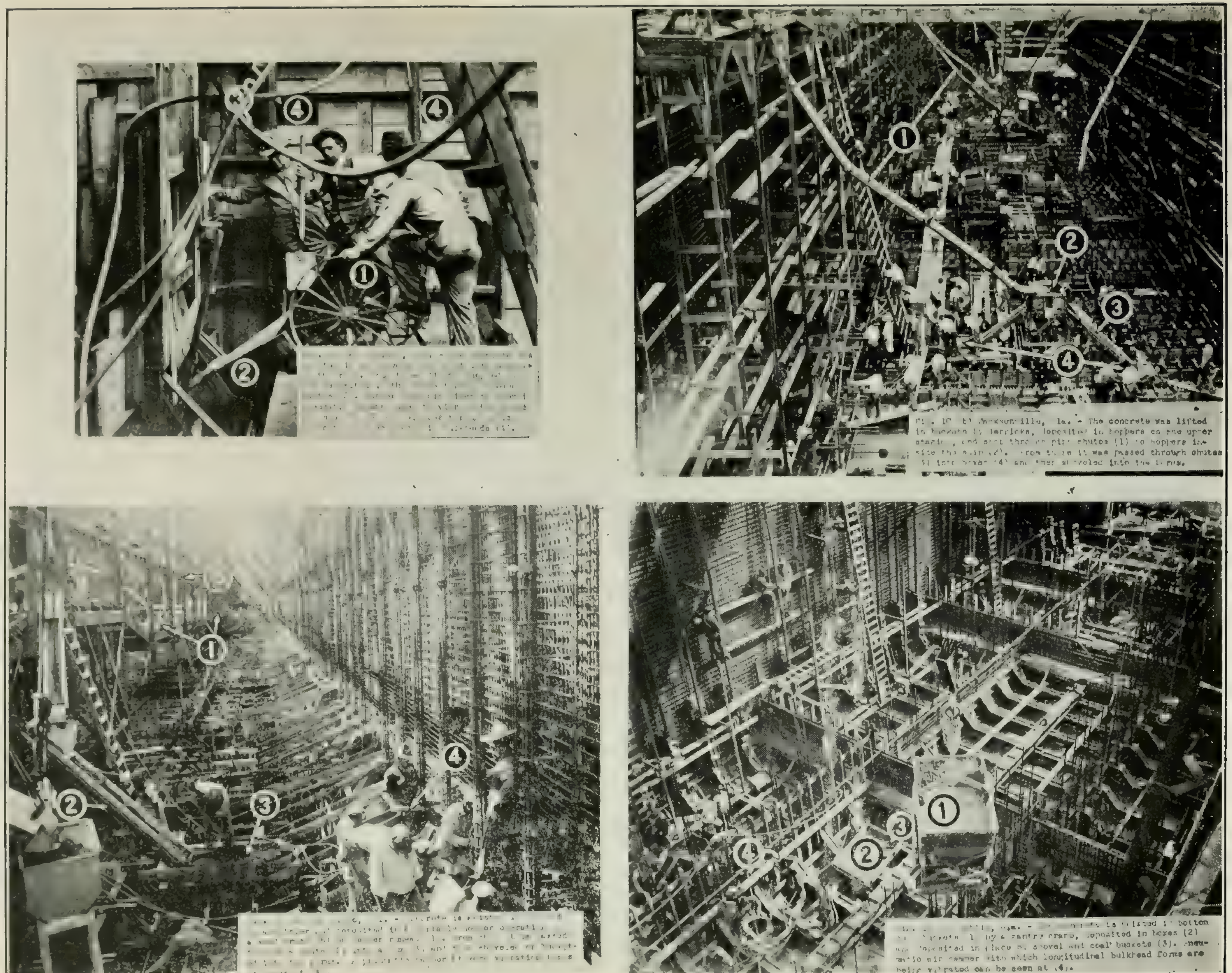


FIG. 10.—CONCRETE PLACED IN THE KEELSONS AND IN THE BOTTOM AND SIDES OF SHEDL AND FRAMES UP TU THE 4 OR 5 FT. DRAFT LINE.

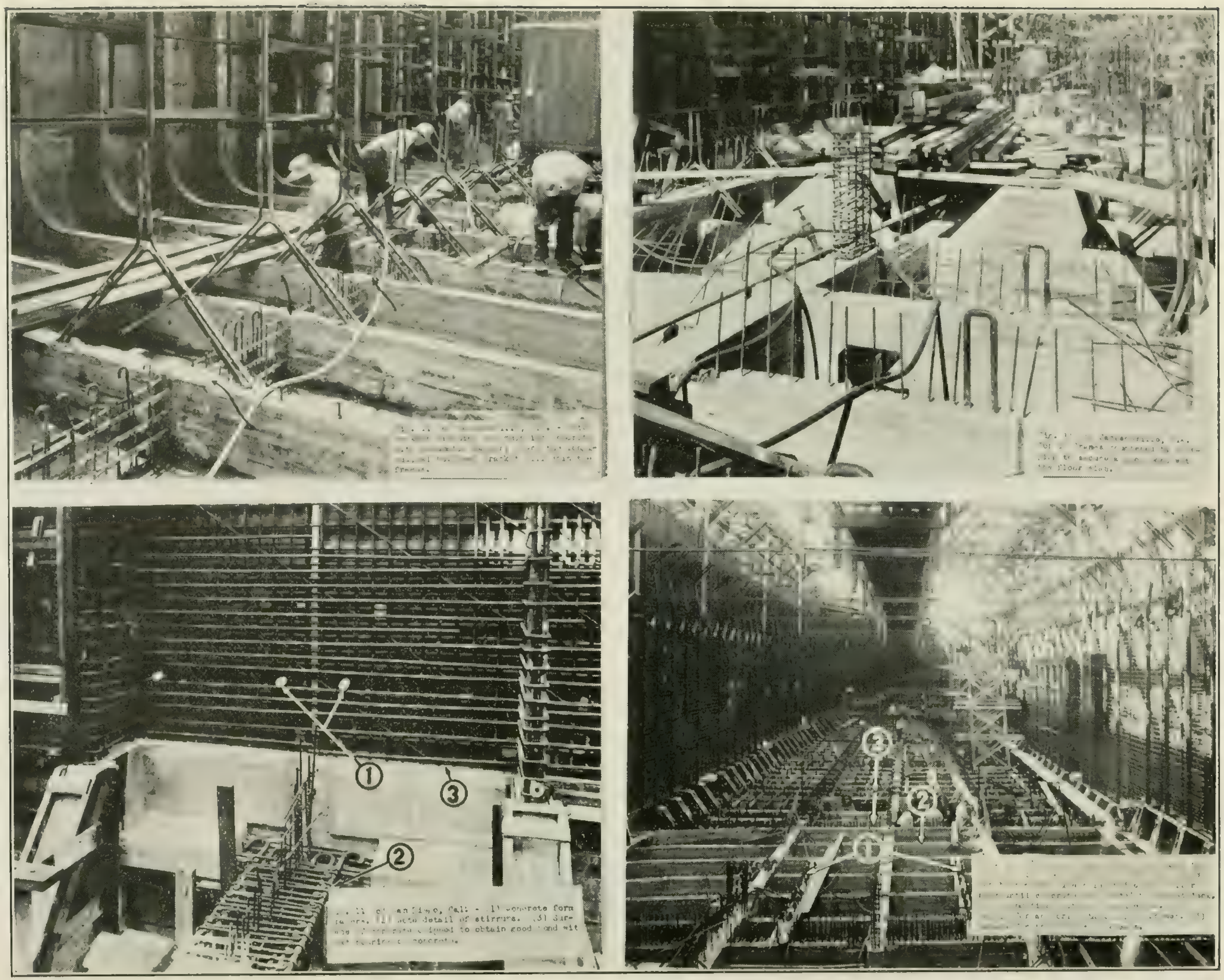


FIG. 11.—BOTTOM INSIDE FORMS REMOVED AND THE CONCRETE POINTED UP WHERE NECESSARY. TOP SURFACE OF CONCRETE WHERE IT WILL JOIN THE SUCCEEDING POUR OF CONCRETE THOROUGHLY CLEANED AND ROUGHENED.

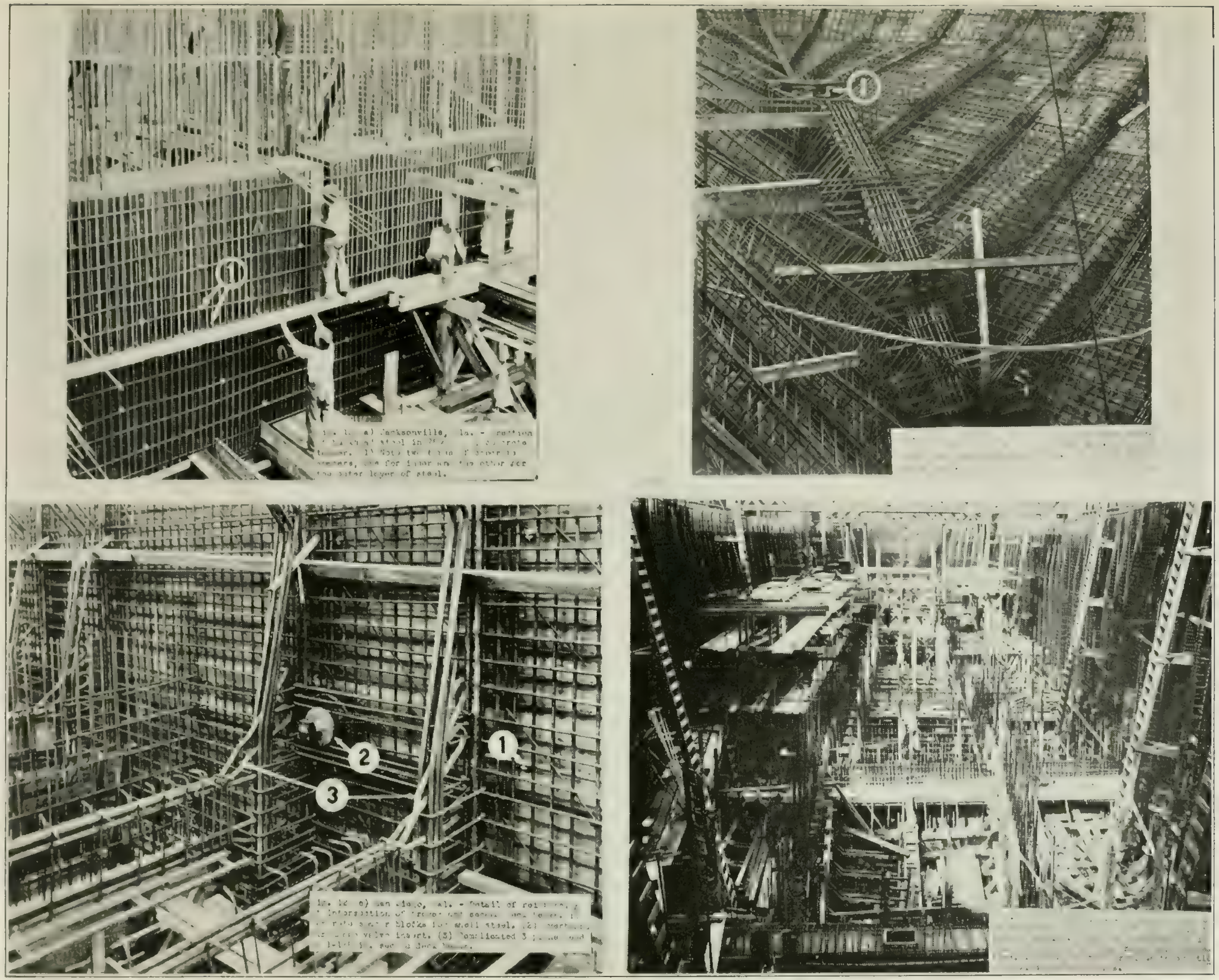


FIG. 12.—ERECTION OF THE FRAME AND BULKHEAD STEEL CONTINUED UP TO THE ELEVATION OF THE SECOND DECK.

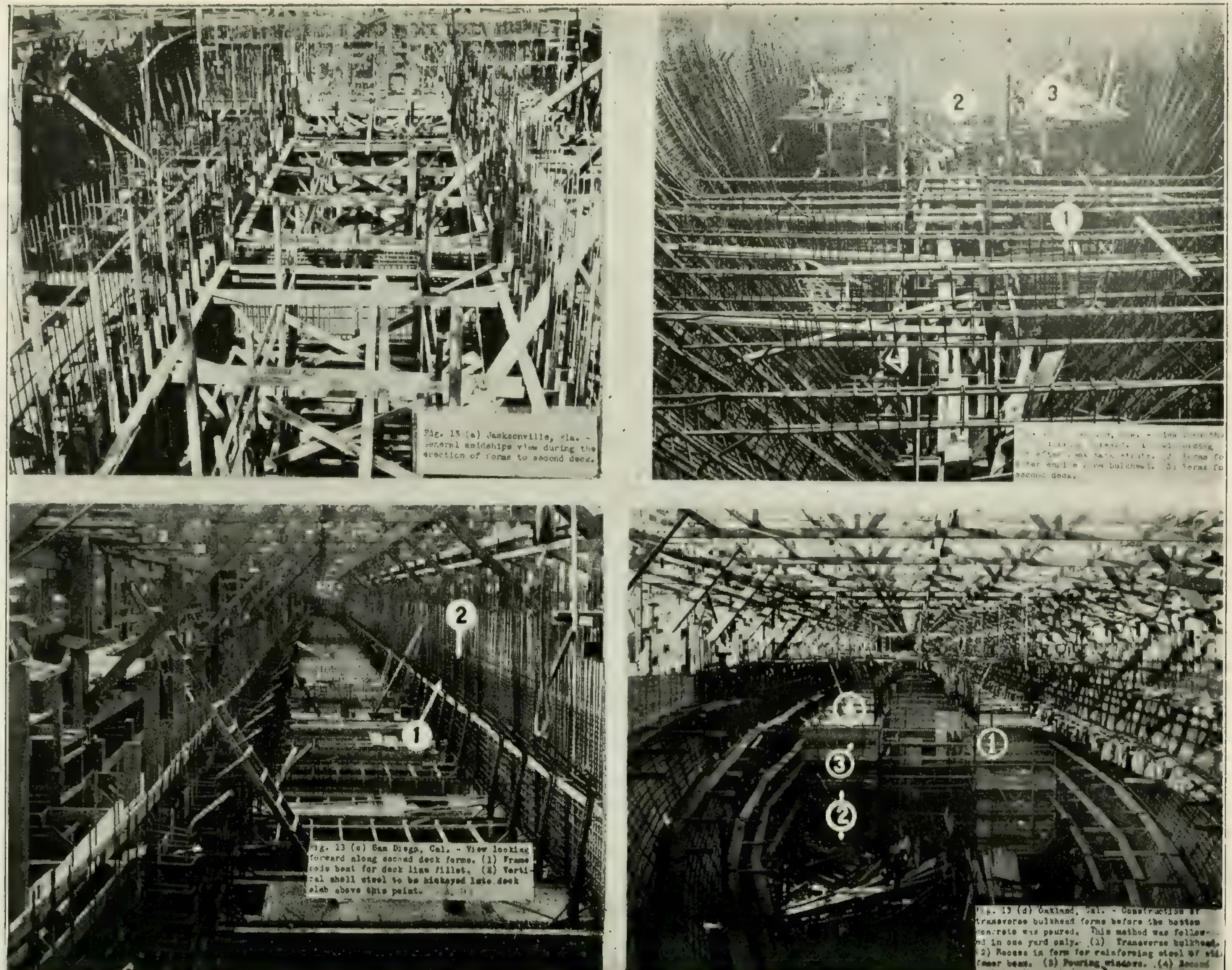


FIG. 13.—INSIDE FRAME, BULKHEAD, AND SHELL FORMS ERECTED UP TO THE SECOND DECK.

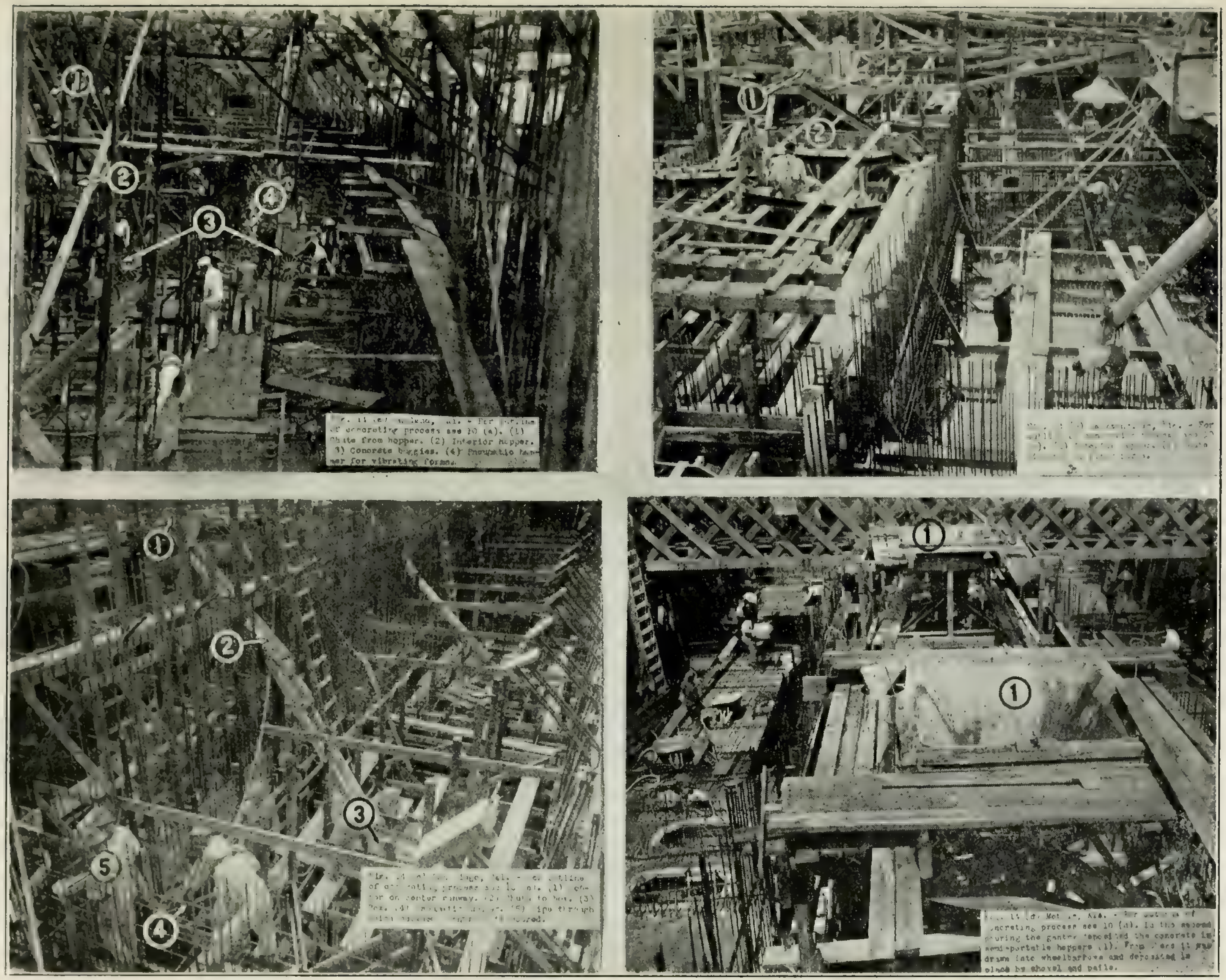


FIG. 14.—CONCRETE PLACED UP TO THE UNDERSIDE OF THE SECOND DECK.

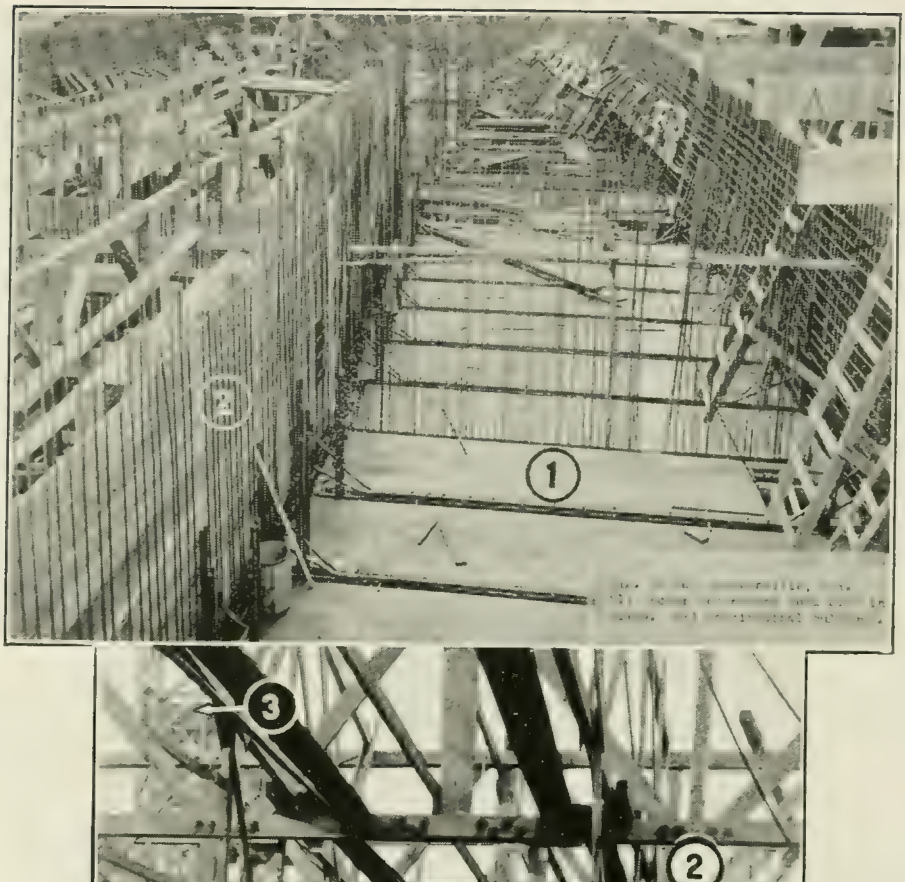


FIG. 15.—FORMS FOR THE SECOND DECK APPEAR AND SLABS PLACED.

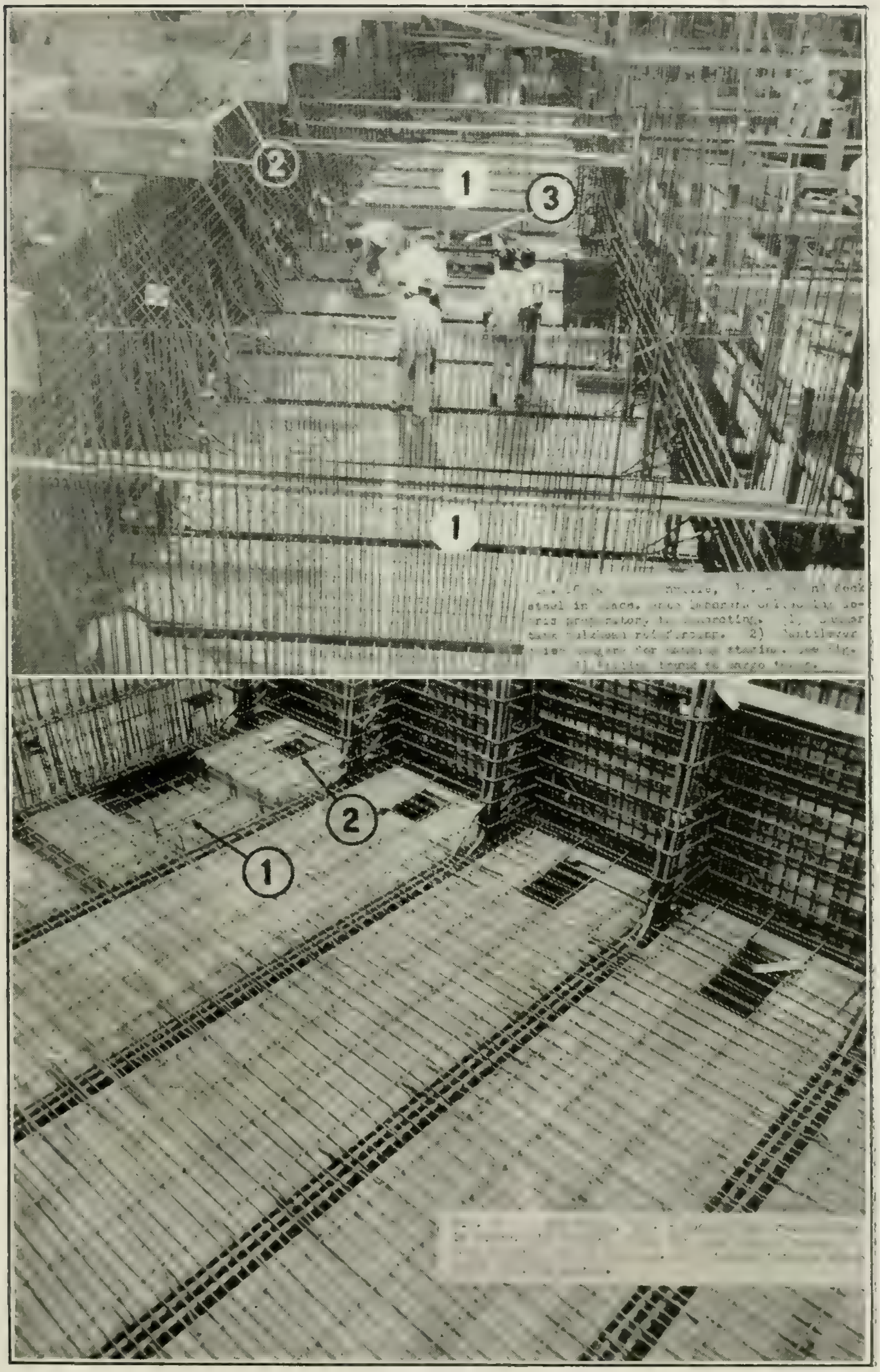


FIG. 16.—REINFORCING STEEL IN THE SECOND DECK PLACED.

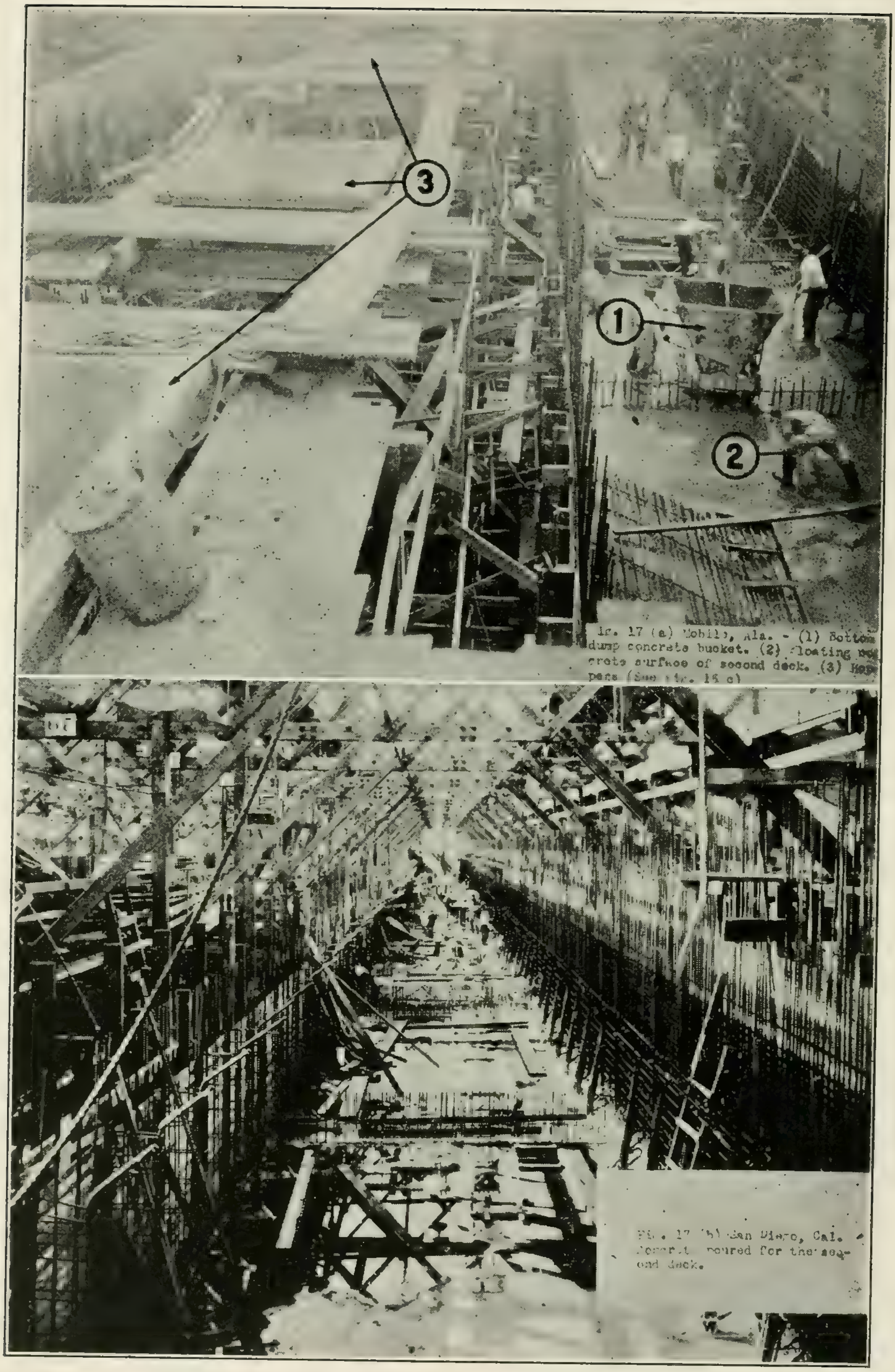


FIG. 17.—CONCRETE PLACED IN THE SLAB AND BEAMS OF THE SECOND DECK.

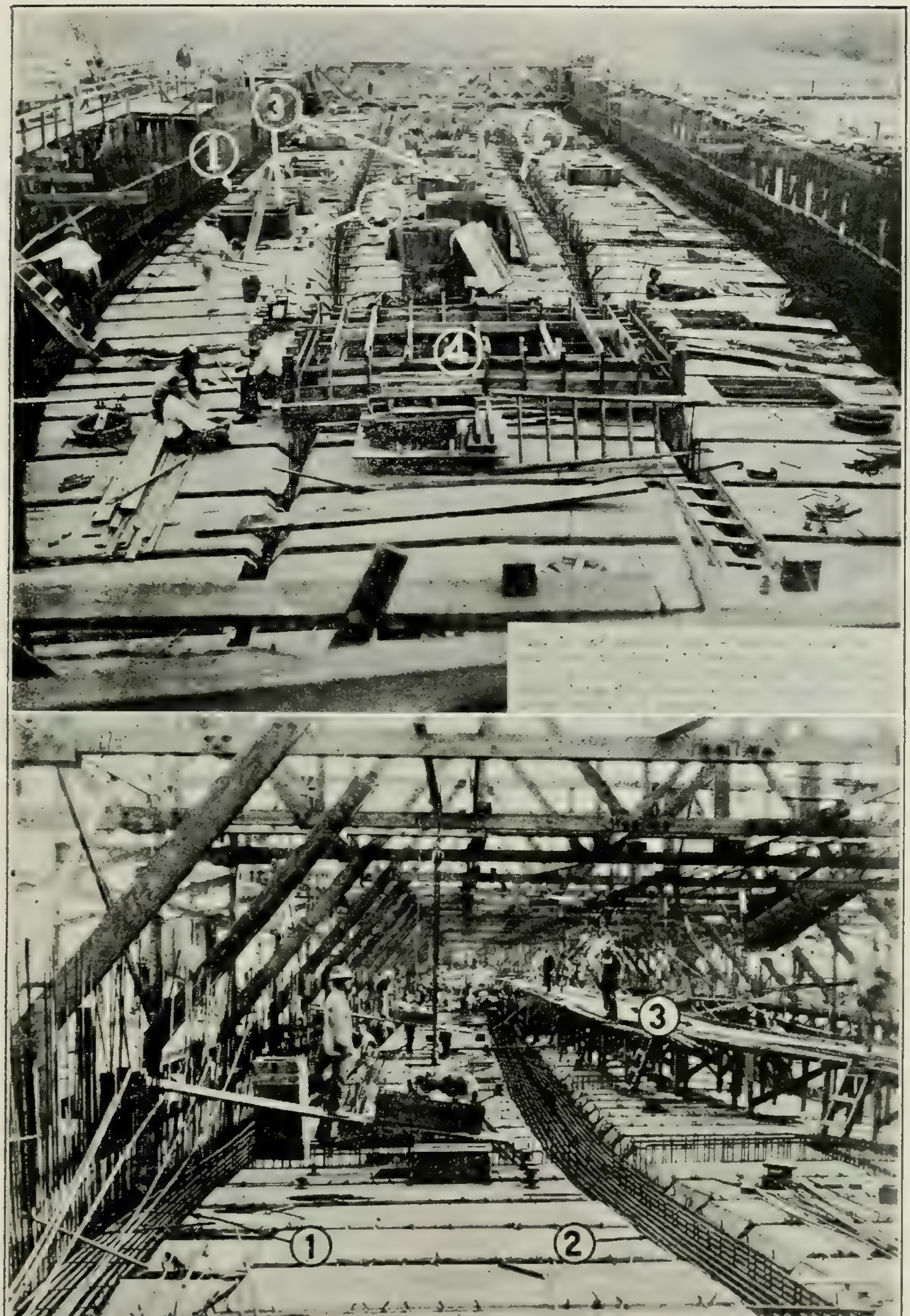


FIG. 18.—REINFORCING STEEL AND FORMS FOR THE INSIDE FRAMES AND BULKHEADS AND FOR THE TOP DECK BEAMS AND SLABS ERECTED.

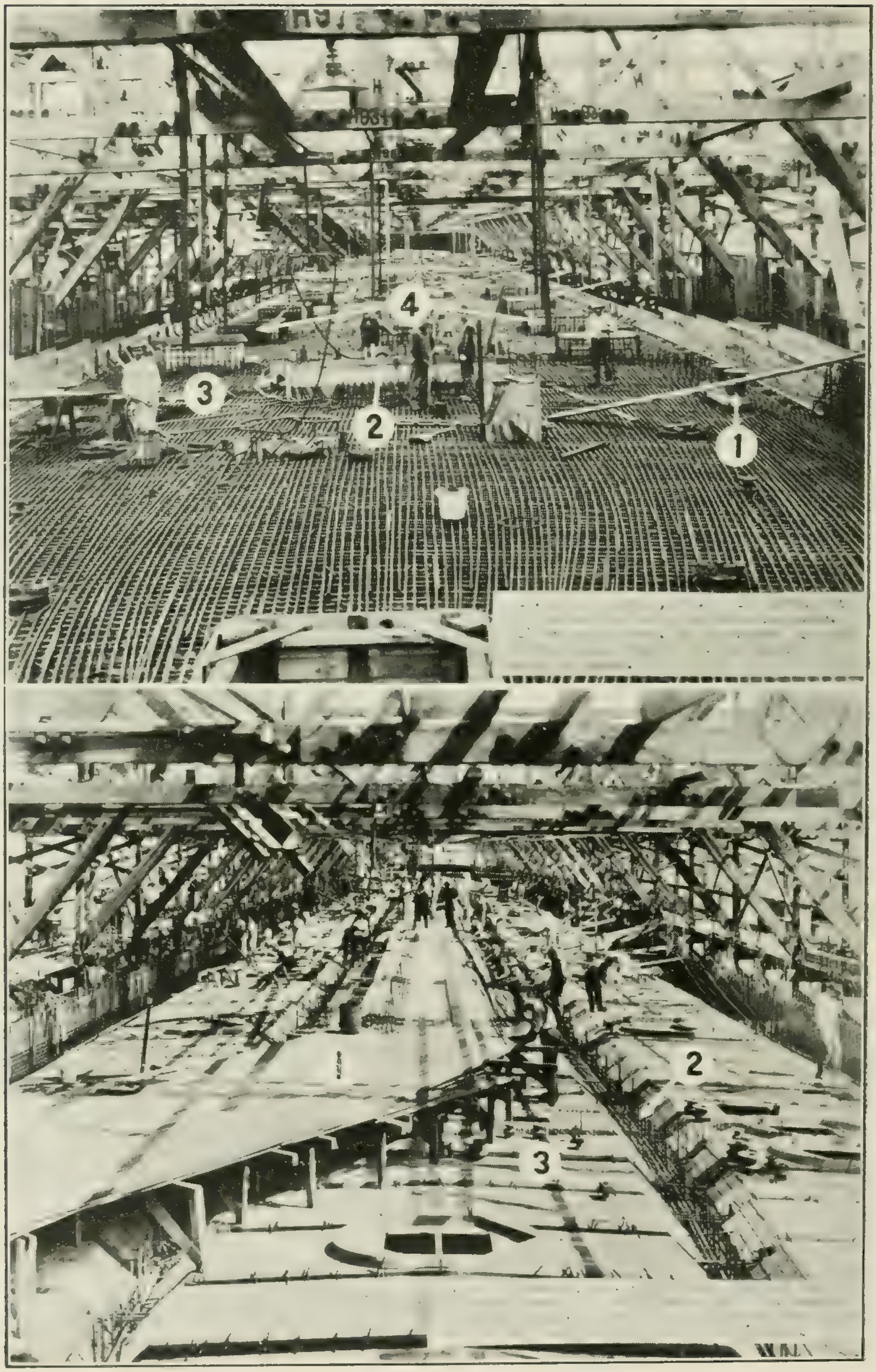


FIG. 19.—REINFORCING STEEL PLACED IN THE FILLET AT THE TOP DECK AND IN THE DECK BEAMS AND SLAB.

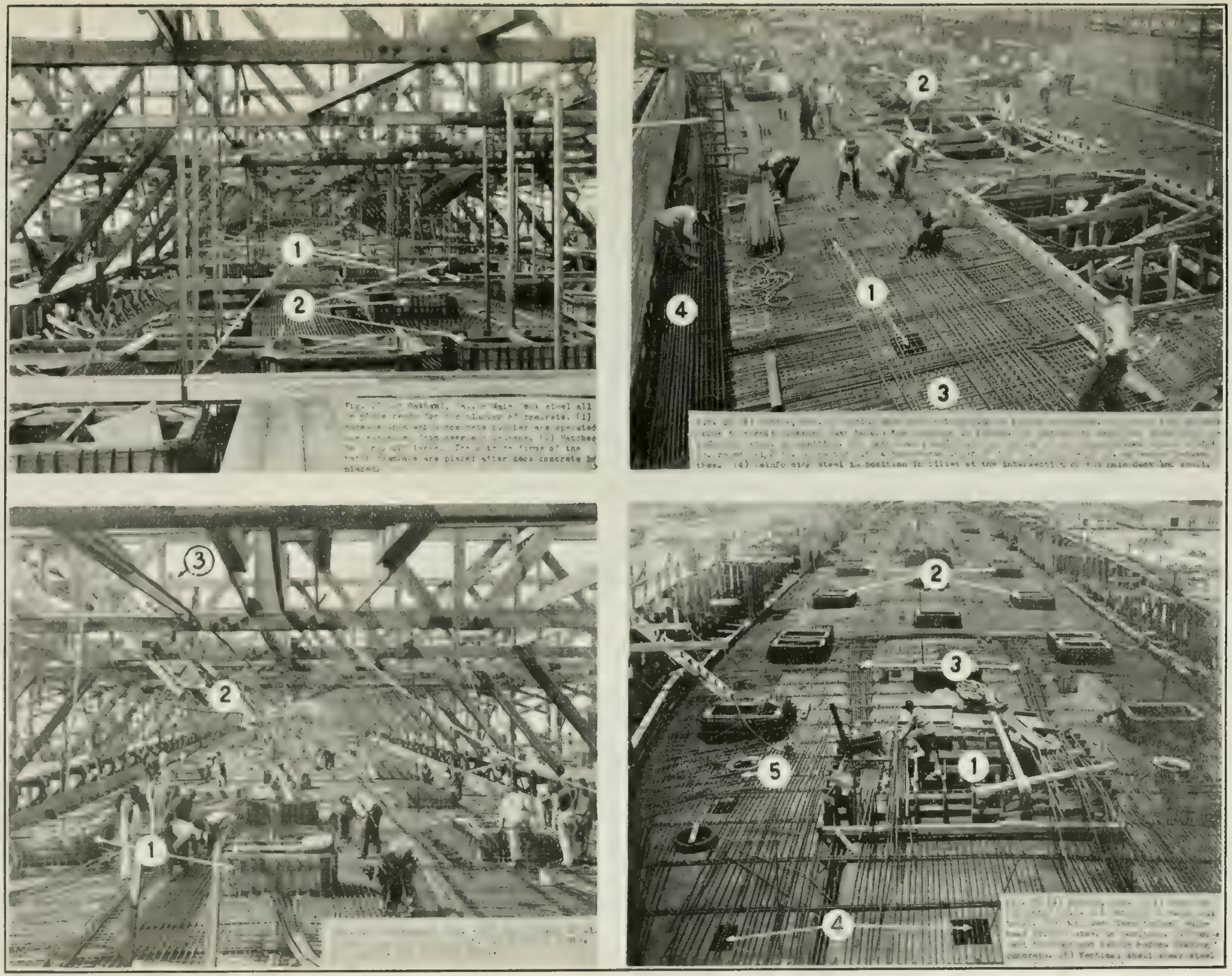


FIG. 20.—TOP DECK SLAB AND HATCH COAMING REINFORCING STEEL PLACED.

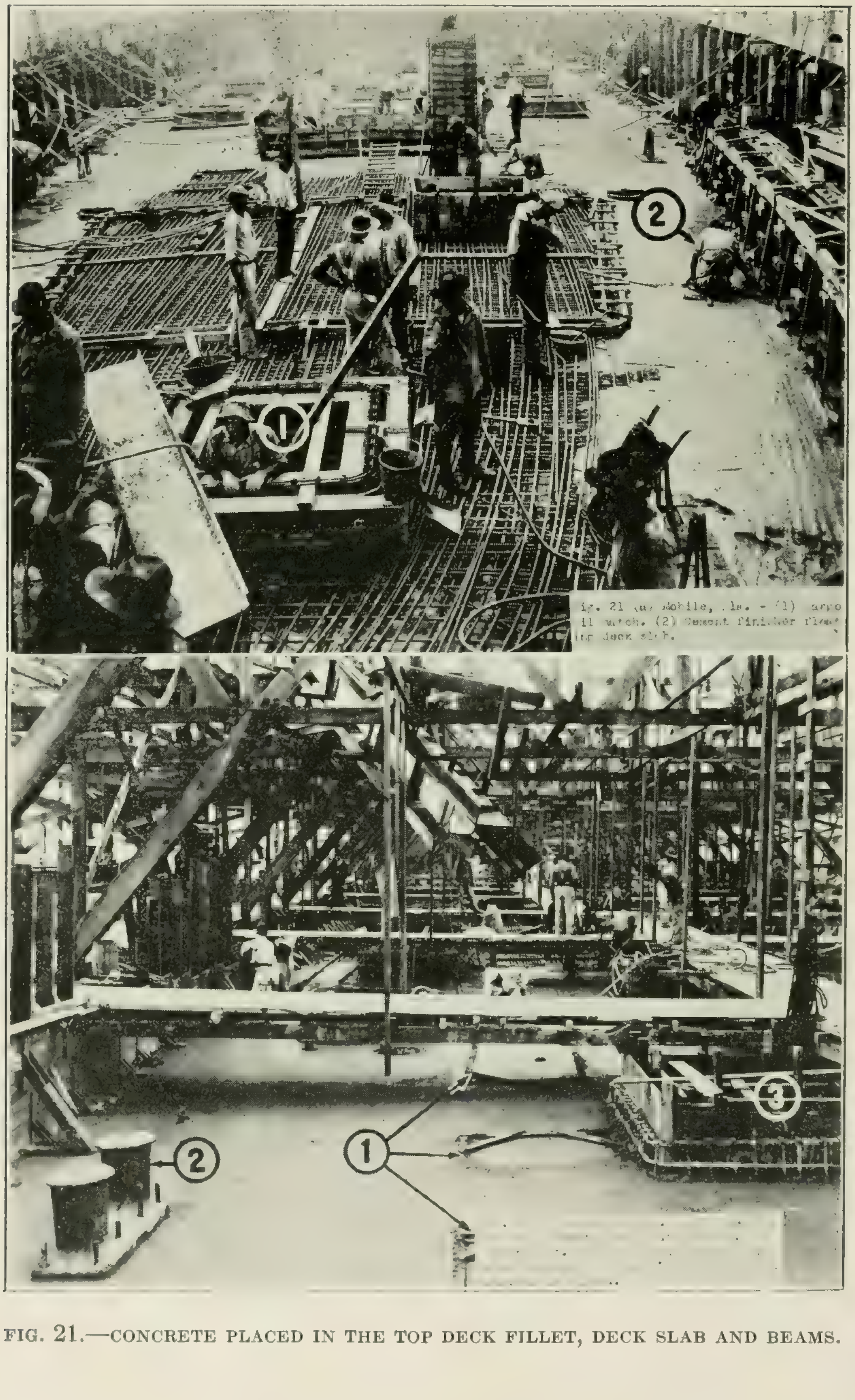


FIG. 21.—CONCRETE PLACED IN THE TOP DECK FILLET, DECK SLAB AND BEAMS.

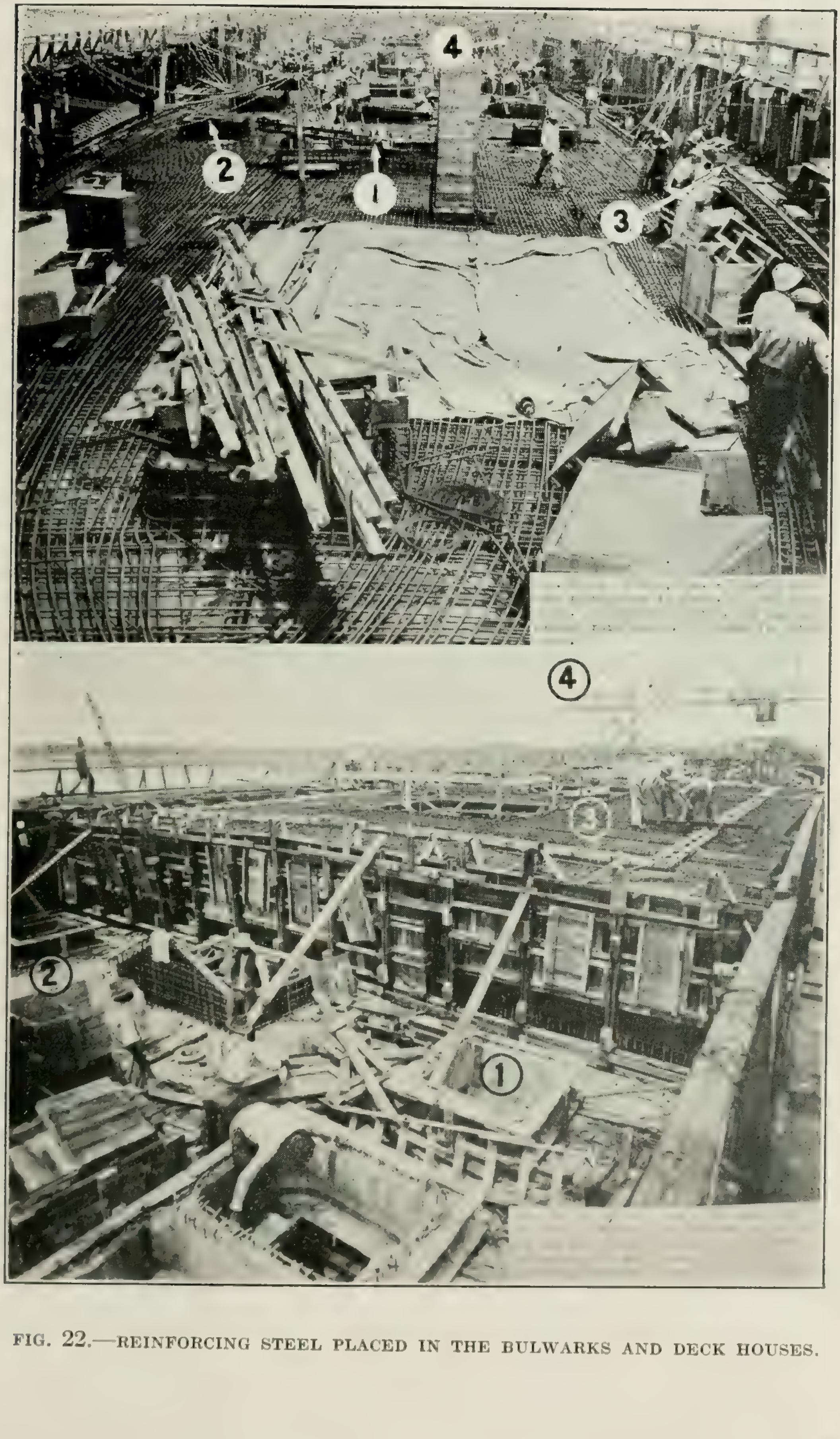


FIG. 22.—REINFORCING STEEL PLACED IN THE BULWARKS AND DECK HOUSES.

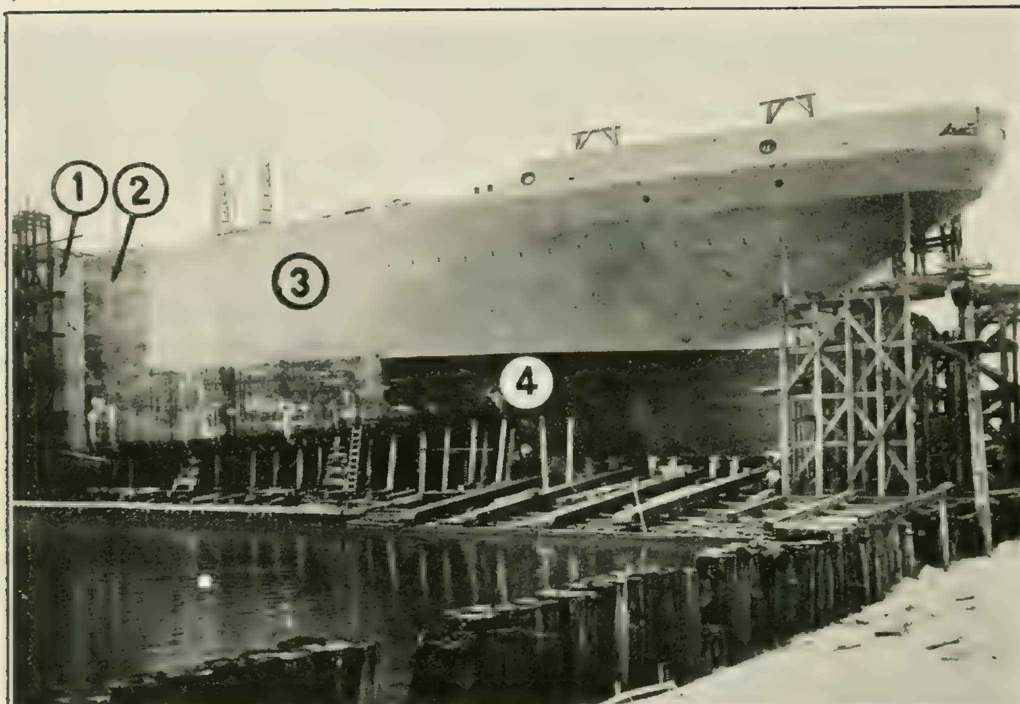


Fig. 23 (a) Mobile, Ala. - Showing different stages of painting outside surface. (1) Concrete surface immediately after forms have been removed. Note scaffolding still in place. (2) The entire surface is given two coats of magnesium fluosilicate and one coat of spar varnish. (3) Above the light draft line two coats of gray enamel are applied over the surface as shown in (2). (4) Below the light draft line one coat of gray enamel (to give body) and a coat of copper antifouling bottom paint are applied over the surface as shown in (2).

FIG. 23.—EXTERIOR, AND IF TIME PERMITS, INTERIOR SURFACES OF THE CONCRETE, PAINTED BEFORE LAUNCHING.

FIG. 24.—LAUNCHING WAYS PLACED IN POSITION UNDER THE HULL, THE BLOCKING CHANGED AND THE HULL LAUNCHED.

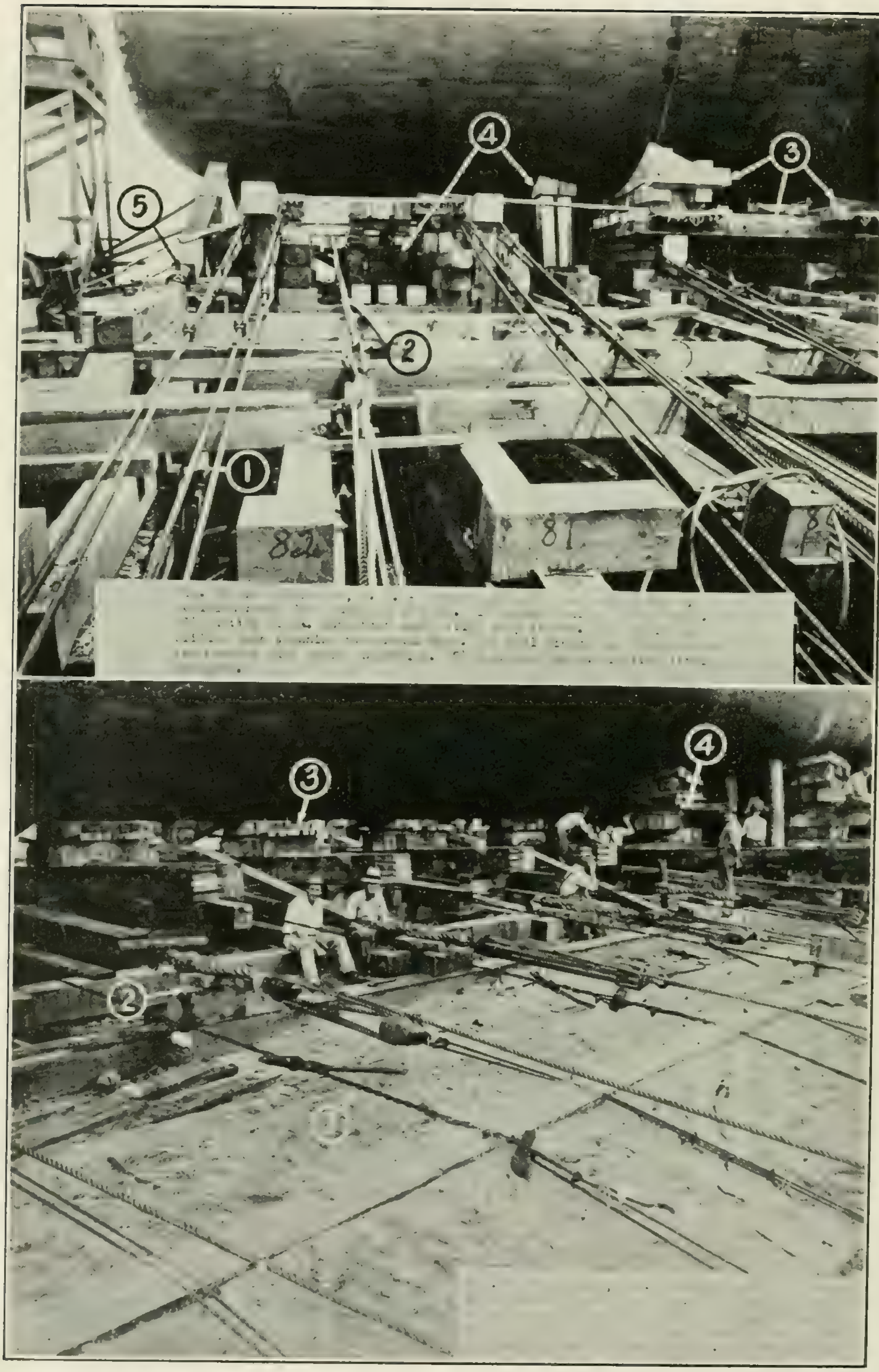


FIG. 25.—INBOARD VIEW OF LAUNCHING DETAILS.

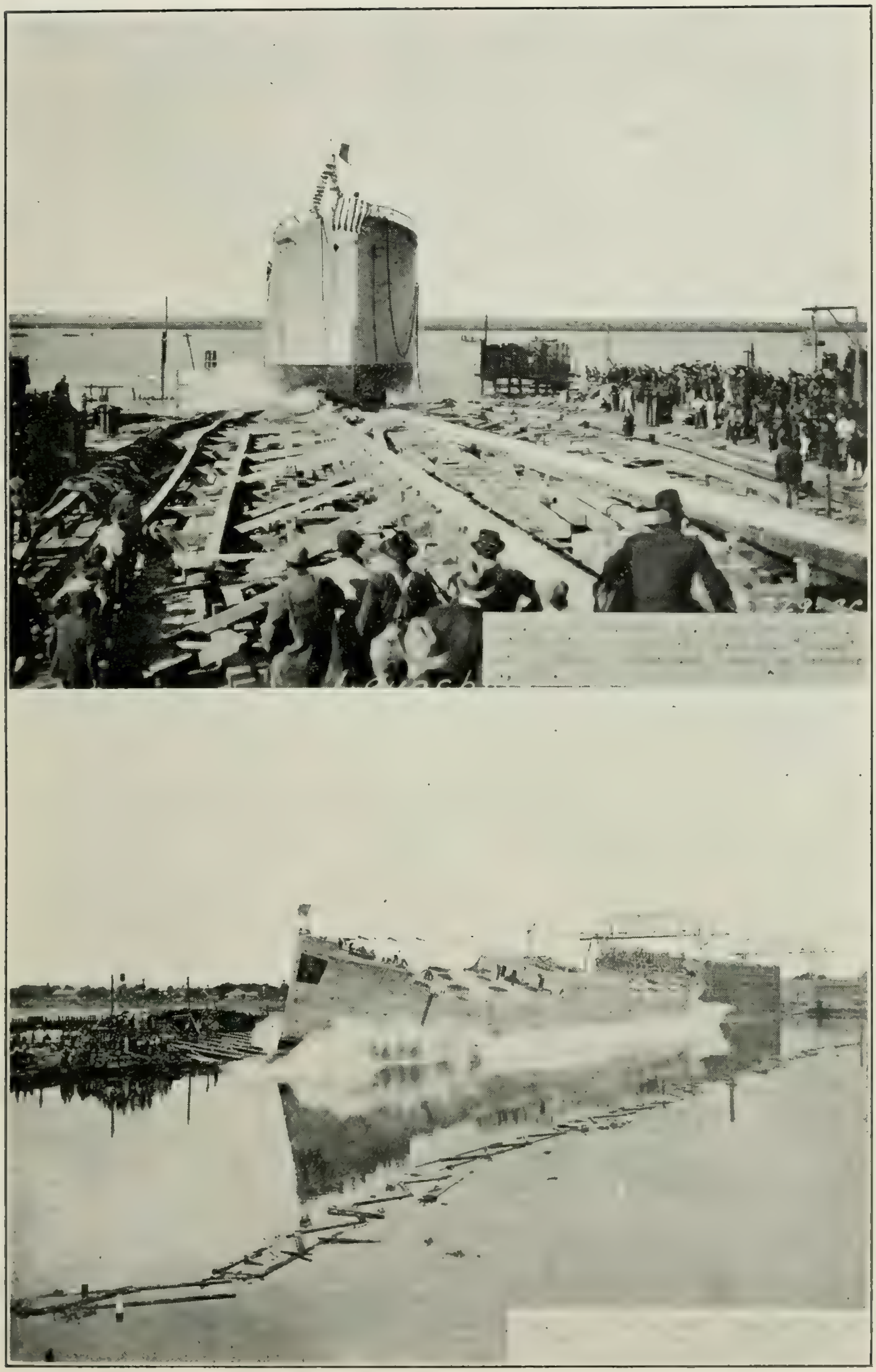


FIG. 26.—VIEWS OF END AND SIDE LAUNCHING.

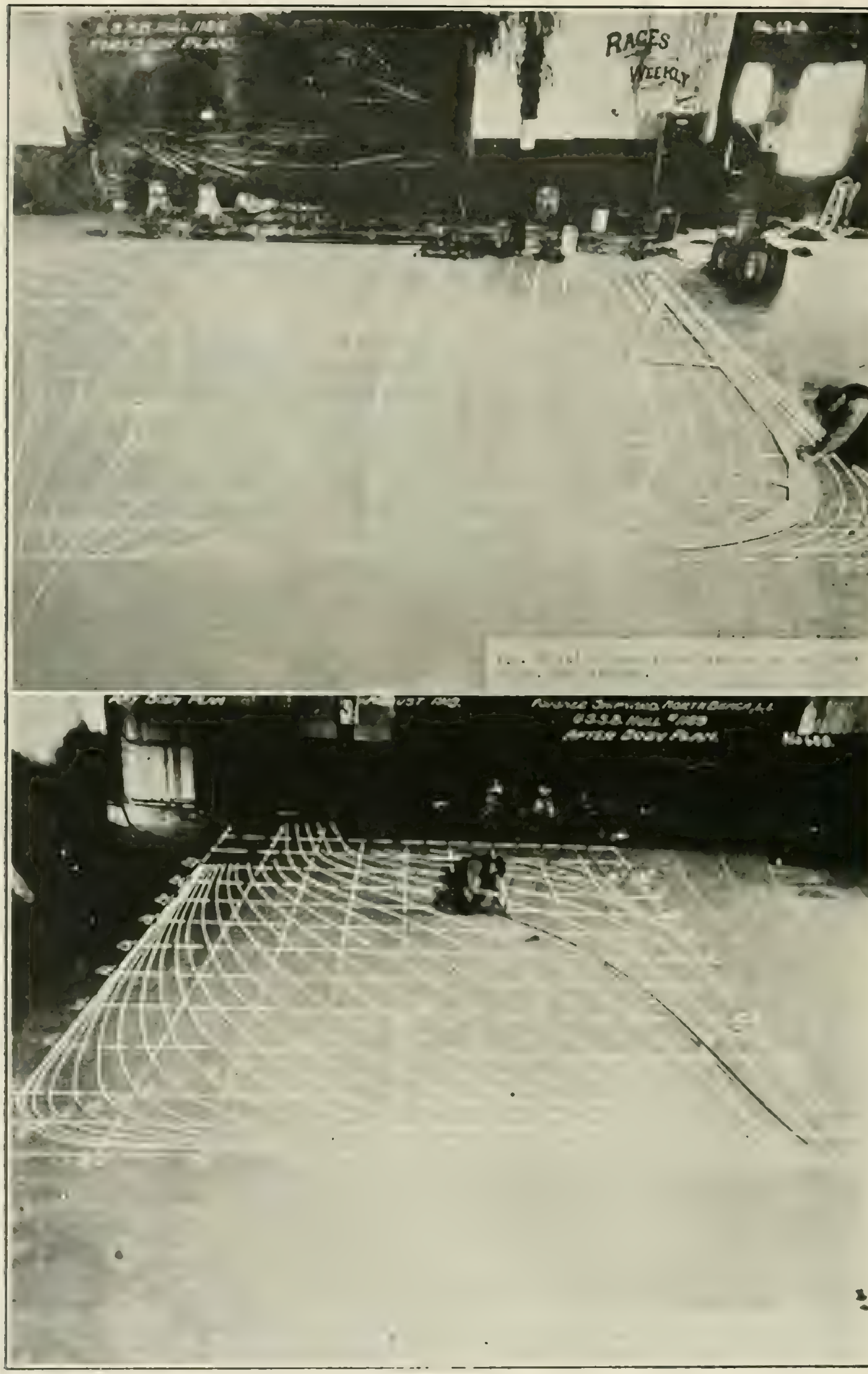


FIG. 27.—VIEWS OF MOLD LOFT FLOOR SHOWING FRAME LINES AND TEMPLATES.

22. Concrete is placed in the shell, bulkheads and frames to the underside of the fillet at the top deck.
23. The upper surface of the concrete which makes a joint with the next pour is cleaned and roughened.
24. The top deck beams and slab forms are placed. (Fig. 18.)
25. The deck inserts for equipment, pipes, etc., are placed. (Figs. 19, 20 and 21.)
26. The longitudinal reinforcing steel is placed in the fillet at the top deck and in the deck beams and slab. (Figs. 19, 20 and 22.)
27. Concrete is placed in the top deck fillet, the deck beams and the deck slab. (Fig. 21.)
28. The reinforcing steel is placed in the hatch coamings, bulwarks and deck erections. (Fig. 22.)
29. The forms are placed for hatch coamings, bulwarks and deck erections. (Fig. 22.)
30. The concrete is placed in all deck erections.
31. All remaining inside forms and staging is removed and all interior surfaces of the concrete are cleaned and pointed. All outside forms on the sides and bottom of the hull are removed and the concrete is cleaned and pointed where defective.
32. All tanks are tested up to the light draft line and pointing and patching of concrete is done if found necessary after testing.
33. The exterior and if time permits before launching the interior surfaces of the concrete are painted. (Fig. 23.)
34. The launching ways are placed in position under the hull, the blocking is changed and the hull is launched. (Figs. 24, 25 and 26.)

It will of course be impossible in this narrative to discuss all the variations or methods which have been employed by each contractor for all operations. Consideration will be confined to the more important and general features.

BLOCKING, SCAFFOLDING, TRUSSES AND OUTSIDE FORMS.

All blocking, outside scaffolding, trusses and outside forms are erected complete before any concrete is poured and in some cases before any reinforcing steel is erected.

The hull as constructed must be supported several feet above the building ways to make the bottom accessible for the removal of forms, examination and painting of the outside of the hull and the installation of the launching ways. In steel ship construction it is common practice to use heavy timber in the form of blocking or cribbing for this purpose. There are three fixed stages of support required: 1. Support during construction. 2. Temporary blocking during the removal of bottom forms and painting. 3. The final blocking on sliding ways for launching.

In concrete ship construction, since a complete flooring or form supported

on joists or stringers must be provided (Fig. 4), it was possible where ways were laid out for side launching to use either a truss or block and crib type of support.

At the Wilmington and Jacksonville yards, the bottom form joists are supported at the proper elevation on bents of prefabricated trusses having upper and lower chords with vertical posts and diagonal bracing as shown in Fig. 4 (b). At Oakland, Fig. 4 (a) a similar truss was used but the floor form sheathing is attached directly to the top chord of the truss. These trusses are placed athwartships. The lower chord is supported upon wedges resting upon the pile caps of the building way and the upper chord carries the form sheathing direct to the joists of the floor forms which run longitudinally with the ship. At the forward and aft ends, the form joists run athwartships and longitudinal stringers are provided between trusses and form joists.

The trusses are divided at the keel into two sections and taper from the bilges of the ship toward the keel to accommodate themselves to the sloping way and the dead rise of the vessel. Sway bracing is provided between trusses at regular intervals. It was necessary that the trusses be so spaced that alternate trusses could be removed to permit the removal of the forms, painting of the bottom and the insertion of the temporary and final blocking. Heavy timber was used in all the yards in the form of blocking or cribbing for the second or temporary support during the removal of the forms and the painting of the hull and also for the final blocking on the sliding ways.

The Mobile and San Diego yards used heavy timber in the form of blocking and cribbing at all stages of support. The details of blocking, however, are not the same in both yards, as can be seen in Fig. 4 (c) and (d). At San Diego girders athwartships are placed on the blocking and support the form joists which extend forward and aft. At Mobile, the girders are placed longitudinally and form the joists athwartships.

The blocking used at San Diego for all stages of support is shown in Fig. 4 (c). On the shipways transverse sills are arranged in groups that center under each alternate transverse hull frame to carry a system of blocking, gang wedges and girders which support the bottom forms during construction. Here the forms are built in sections consisting of removable panels of a uniform width and extend over two frame spaces where their fore and aft edges rest upon transverse girders. The girders for each section are supported on successive sets of blocking and are held to their proper elevation by adjusted gang wedges. At the forward and aft ends the bottom forms are not built in panels but as a unit.

Scaffolding is necessary to support the outside forms. The details of the design of the scaffolding differed somewhat in the several yards. The general scheme employed, however, was quite similar. The scaffolding used at Oakland and San Diego is shown in Fig. 4 (a) and (c). Essentially the scaffolding consists of two lines of trestles built of light wood construction which are anchored at the bottom to the sills of the building ways, and at the top, at regular intervals, they are tied and held apart across the ship by trusses of similar construction. In some cases these top trusses are placed quite close together (10 to 15 feet c to c) while in one yard (Fig. 5 (b)) only three

trusses were used for the entire length of the ship (435 feet). The wide spacing of the trusses affords less interference with overhead handling of materials into the hulls and is to be preferred. Struts from the scaffolding held the outside forms in place. (Fig. 5, Item 4.)

The forms for ship construction must be smooth, very rigidly built and braced, and more exactly constructed than concrete forms for ordinary building construction. The concrete sections have been made a minimum in size which will accommodate the steel and meet the strength requirements, and any deviation in the form construction either results in added weight of hull, lack of space or cover for the reinforcement or lack of uniformity in the concrete surfaces.

The hull lines to which the outside forms had to be constructed were obtained from the mold loft floor (Fig. 27) by one of several methods. Either templets were made of the cross-section required for the midship body and every frame of the forward and aft body, and the forms constructed in place from these templets, or the forms were prefabricated in panels, the lines being taken directly from the mold loft floor, or portions of the studding were cut to shape in the mold loft, thus giving the ship lines when erected to which the sheathing was directly applied. In most yards a combination of these methods was employed.

The thin board form of templet commonly used in steel ship construction was employed in all but one case where templets were used. At Wilmington, N. C., an adjustable templet was devised and successfully used.

This templet consists of a flexible batten attached to a rigid frame in such a manner that the batten can be bent to any shape desired and rigidly supported in that position simply by tightening the bolts which pass through strips attached at their ends to the flexible batten. With these "curved sets" as they were called it was possible to lift lines from the mold loft floor and transfer them to the steel bending tables or to the form timbers. For further use the lines were transferred to heavy sheets of templet paper.

Cypress was largely used as outside form lumber in the East and Oregon pine in the West. Since the outside forms are exposed to the weather for several weeks or months before the concrete is poured it is necessary that some material be used which will not warp badly. Most yards used light $\frac{7}{8}$ or $\frac{3}{4}$-inch sheathing throughout. One yard used $1\frac{3}{4}$-inch sheathing in the bottom and bilge forms.

It was found to be impossible to secure the outside forms rigidly in position so that they would not spread and would withstand the vibration of the air hammers which were used in placing the concrete unless provided with a through tie to the inside forms. Another method is to tie both inside and outside forms to the reinforcing steel.

In one case the outside forms were prefabricated in panels throughout the entire ship. Fig. 5 (d). In all other cases, the outside forms were prefabricated in panels only through the middle body portion of the ship and the curved portions forward and aft were built as a unit in place. This latter method has proven the more satisfactory.

At San Francisco (Fig. 4 (a)) and Wilmington the entire outside form was

built as a unit in place but joints were made in the bottom sheathing for removal in panels through the middle body section. A description of the method employed is given below. Its construction was comparatively simple. Shiplap $\frac{3}{4}$ in. in thickness was used horizontally, backed by 2 x 6 in. studs at the frames and midway between frames or on $25\frac{1}{2}$-in. centers. The studs were fitted to the proper shape on the mold loft floor, allowance being made

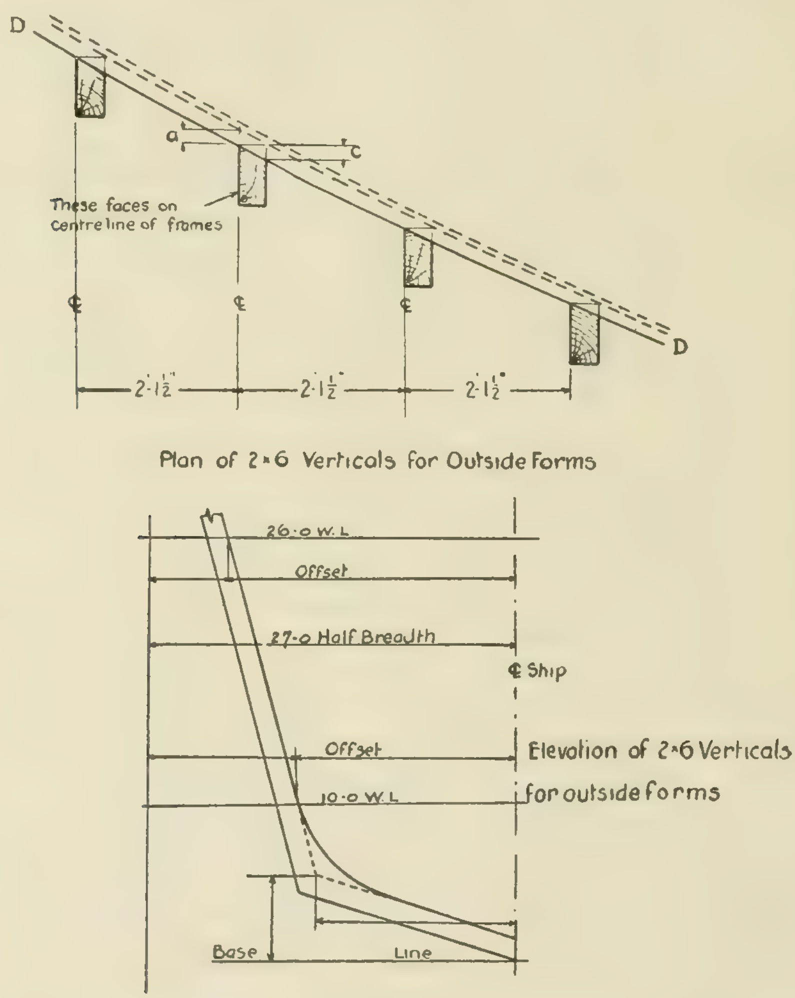


FIG. 28.—DETAILS OF 2 BY 6-IN. VERTICAL STUDS FOR OUTSIDE FORMS.

for the thickness of the shiplap. They were erected in the hull so that one face was coincident with the centerline of the frame (Fig. 28).

For purposes of erection, wires were strung along the center line of the hull and also 27 ft. on either side of the center line, 27 ft. being the half breadth of the ship. The base line was also given at each frame. Points for stringing these wires were given by the surveyor.

While the ribs were on the mold loft floor, the 36-ft., 26-ft. and 10-ft. water lines were scribed on them, and also the intersection of the bottom and

sides located as in Fig. 28. The offsets at each water line from both the center line and the 27-ft. line were scribed on a batten at the mold loft, four battens being required. These battens were then used in setting the studs, the offset from the 27-ft. side being used and the offset to the center line used as a check. A light batten "D" was fitted across the faces of the studs, as in Fig. 28, to take the curve of the ship so that the distance "a" could be measured and set off on the stud as distance "c." This done at several places vertically on the studs, gave points on a line so that they could be added to the proper surface "DD" ready to receive the shiplap.

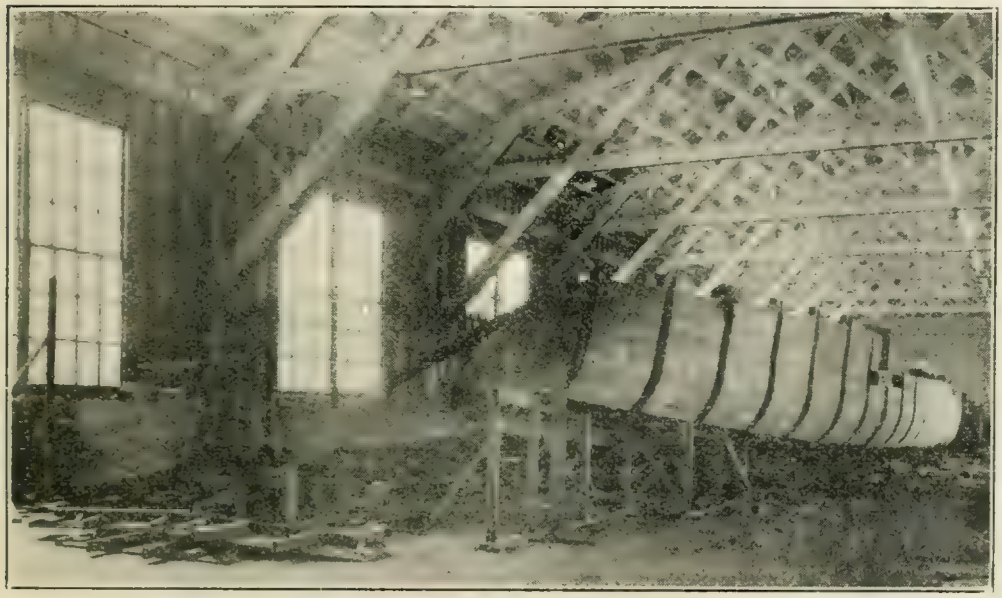


FIG. 29.—SECTION OF FRAME AND BILGE FORMS ERECTED IN THE MOLD LOFT FOR FITTING.

INSIDE FORMS.

With one exception all the contractors prefabricated the inside forms in sections ready to set into position without requiring any carpenter work to be done within the hull. The lines for the inside forms were obtained from the mold loft floor or from templets taken from the floor. These forms had to be constructed with great exactness, for in many cases there was only ⅜ in. clearance from the steel on all sides. A group of inside bottom frame and bilge forms assembled in the mold loft is shown in Fig. 29. To show the accuracy with which such forms can be made, it is of interest to know that in the case of one ship none of these prefabricated forms had to be removed for adjustment after being set in place.

There perhaps is no phase of the concrete ship construction work which is more complicated than the construction of the inside forms. A description of the exact method followed at one of the yards in obtaining the shape and dimensions of the various sections will be of interest.

The frame drawings gave the thickness of the shell and the depth and width of the frame, as in Fig. 30. The problem was to obtain the width of the frame forms N, W, and the panel form P and also their relations to the mold loft line which passes through the point ML in Fig. 30 (c). The bevels across the frames at the critical places were first obtained. These critical places were at the bottom of the bilge "L," top of the bilge "U," and at the

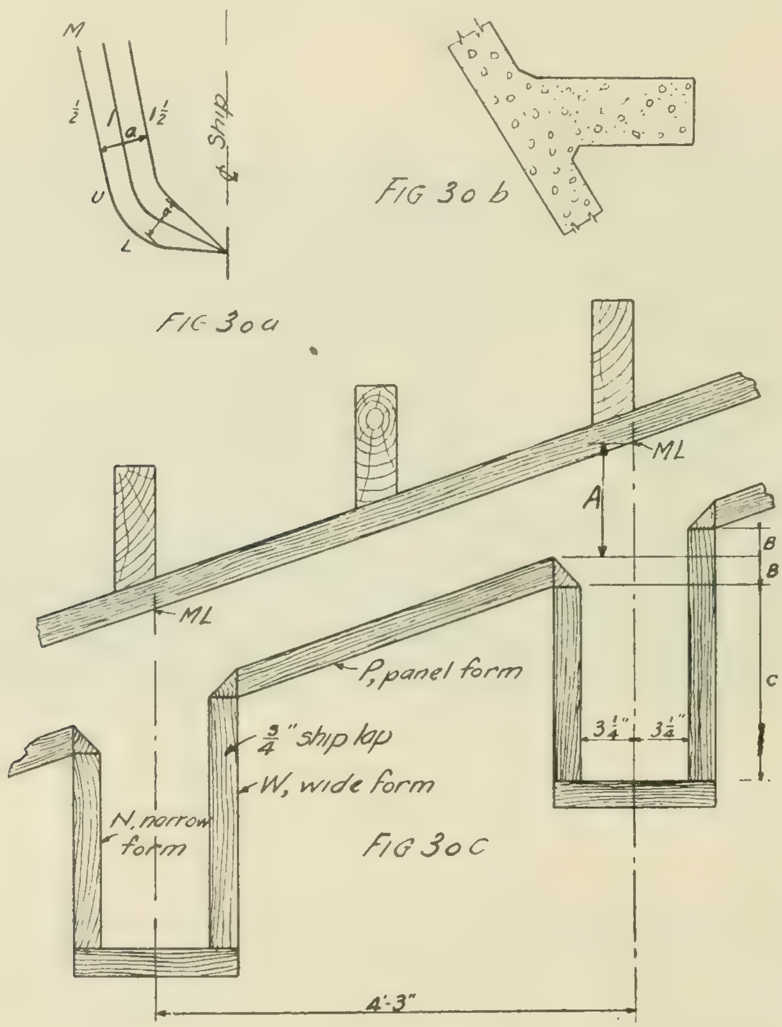


FIG. 30.—DETAIL OF FRAME FORM PANELS.

main deck. The method of obtaining the bevels was to measure the distance "a" in Fig. 30 (a) between half frames on the mold loft. This, divided by the frame interval, 51 inches, gave the bevel across that particular frame between the two half frames. These bevels were taken in a plane perpendicular to the shell.

The next step was to obtain the dimensions A, B and C of Fig. 30 (c) which were all functions of the frame bevels. As a great number of these had to be computed, graphical charts were made for convenience.

It will be noticed in Fig. 30 (c) that the diagonal distance "A" is given to the outside line of the shiplap form produced. This was done in order that a right-angled beveled chamfer strip could be used at the intersection of the frame form and panel form as shown. The panel forms were so cut that their edges were in the same plane as the side of the frame.

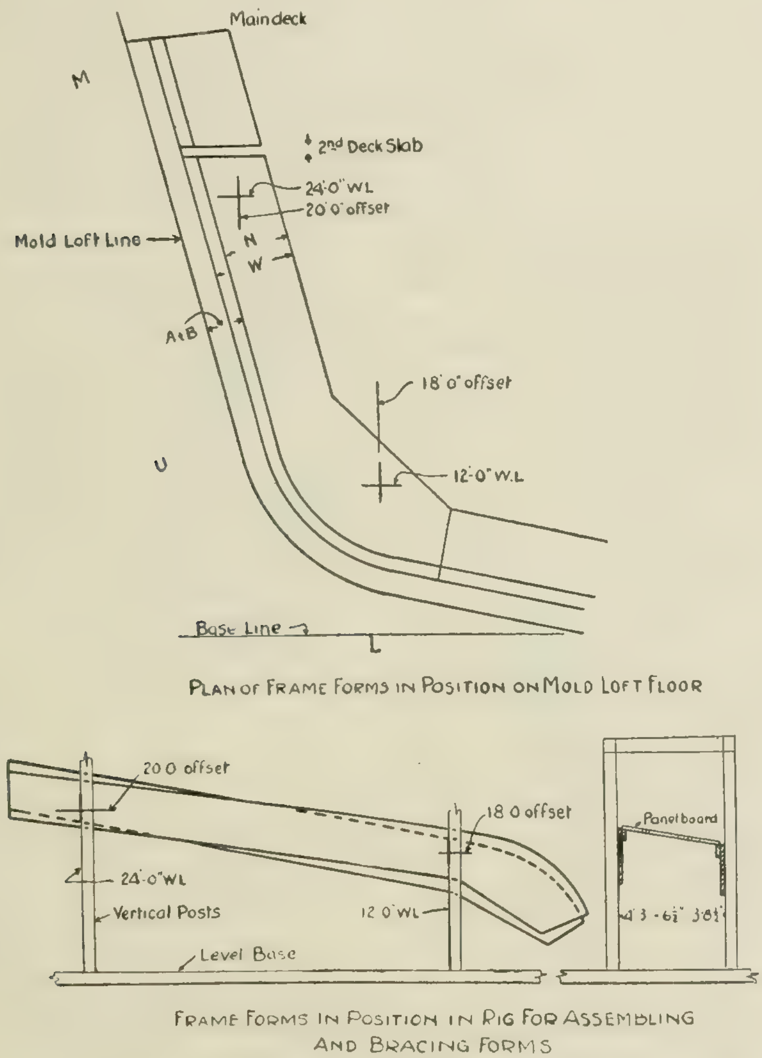


FIG. 31.—PLAN OF FRAME FORMS IN POSITION ON MOLD LOFT FLOOR, AND RIG FOR ASSEMBLING AND BRACING.

Following is the form in which the information was given the mold loft carpenter.

Frame.	Lower Bilge.	Upper Bilge.	Main Deck.
14 A	6-1/8	5-1/4	4-7/8
14 B	1-3/4	1-1/8	7/8
14 C	1-11	1-10-1/4	1-10

As the mold loft lines were straight, i. e., not the conventional ship curves, forms for the sides "N" and "W" could be made up to approximate size before hand. These would be brought in to the floor in pairs and laid down adjacent to the frame line for which they were intended. It was then a simple matter to apply the dimensions A, B and C at the critical points M, U and L, Fig. 30 (a) and obtain points which when connected gave the proper width of form for the entire length. At this stage all brackets and beams framing in to the frames as shown on the frame detail were provided for, so that the measuring and fitting in the ship were reduced to a minimum.

The next step was to assemble the "W" of one frame with an "N" of another and connect them with the panel forms "P." Fig. 30 (c). Before the forms N and W were lifted from the floor, water lines and offsets in even feet were scribed on, for example, in Fig. 31 (a) the 12-ft. and 24-ft. water lines and 20-ft. and 18-ft. offsets. This, of course, co-ordinated them precisely in two dimensions and gave the proper position which they would occupy in the ship. There only remained to place them in some sort of a rigid rig which would maintain them in their proper relation, the correct distance apart, while the panel boards "P" were being nailed on and the whole form braced so that it could be handled and erected.

Fig. 31 (b) shows sketches of this rig. The panel boards were nailed to cleats on the sides of the forms in approximate length. They were then sawed so their edges were in the same plane as the side of the frame, so that the right-angled beveled chamfer strip could be nailed on.

It is worthy of note that the forms built up in this way fitted the varying bevels of the shell with great accuracy and no trouble was experienced in maintaining the proper shell thickness at all places.

INSERTS.

There are from 5,000 to 6,000 so-called "inserts" or fittings which must be cast into the hull of the concrete ship as the concrete is placed. These are composed largely of anchor bolts, sockets, pipe sleeves and miscellaneous holes which must be provided for attaching equipment and furnishing openings for pipes, drains, etc. There are also many large inserts which must be provided such as stern frame, stem plate, hawse pipes, sea chests outboard discharge fittings and the like. (See Figs. 32, 5, 6, 7, 19 and 21.) Large fittings like the cast steel stern frame and stem shoe were anchored to the concrete by means of bars which were bolted or riveted to the fittings and the free ends of the bars were cast into the concrete. There was a wide difference of opinion as to the type of anchorage which should be provided to secure the stern frame to the hull and certain variations of the construction of this detail were permitted as shown in Fig. 32. Sea chests and all other large pipe connections were provided with large flanges on either face and the reinforcing bars were placed between the flanges which were later filled with concrete (Figs. 6 and 12), the faces of the flanges being in a plane with the surface of the finished concrete wall.

Wherever possible the inserts were placed on the forms immediately after

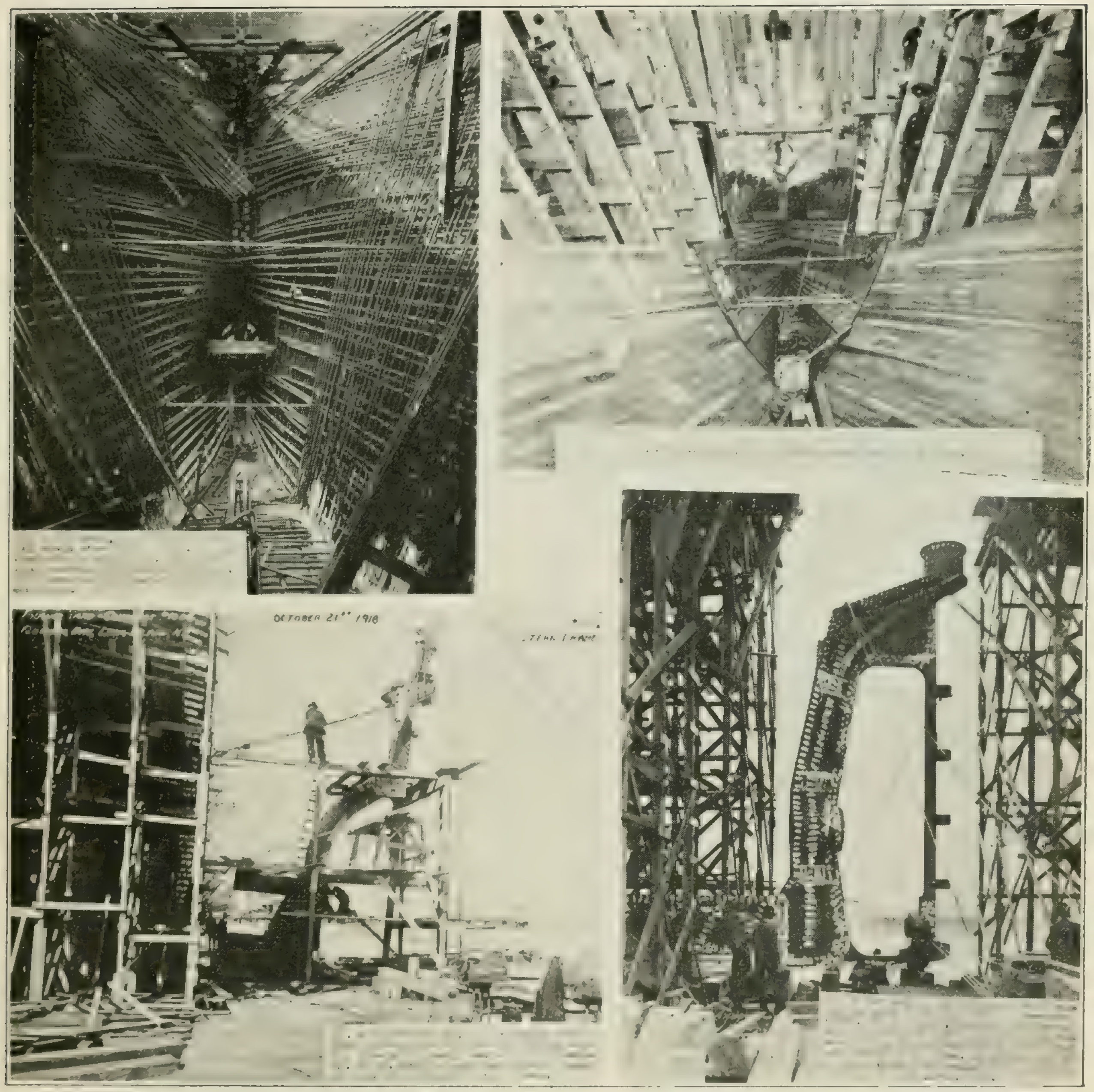


FIG. 32.—TYPES OF STERN FRAMES USED ON CONCRETE VESSELS.

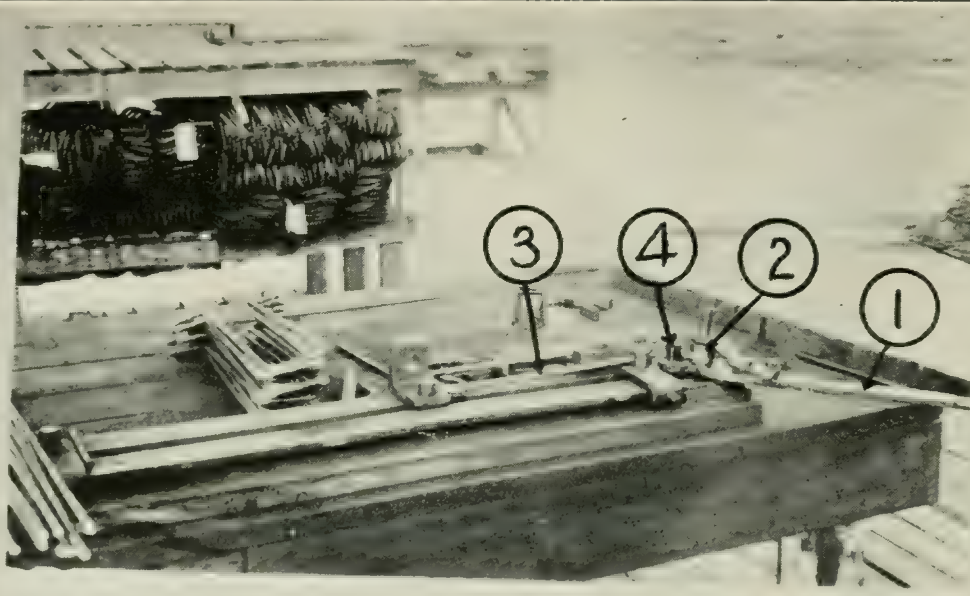


Fig. 33 (a) Hand Operated Stirrup Bender. (1) Lever arm which rotates roller pin (2), which in turn bends stirrup rod (3) about fixed pin (4).

Fig. 33 (b) Air operated stirrup bender used at Jacksonville. The bending is done as in 33 (a), the hand operated lever being replaced by the air operating device.

FIG. 33.—MACHINES FOR BENDING REINFORCING STIRRUPS.

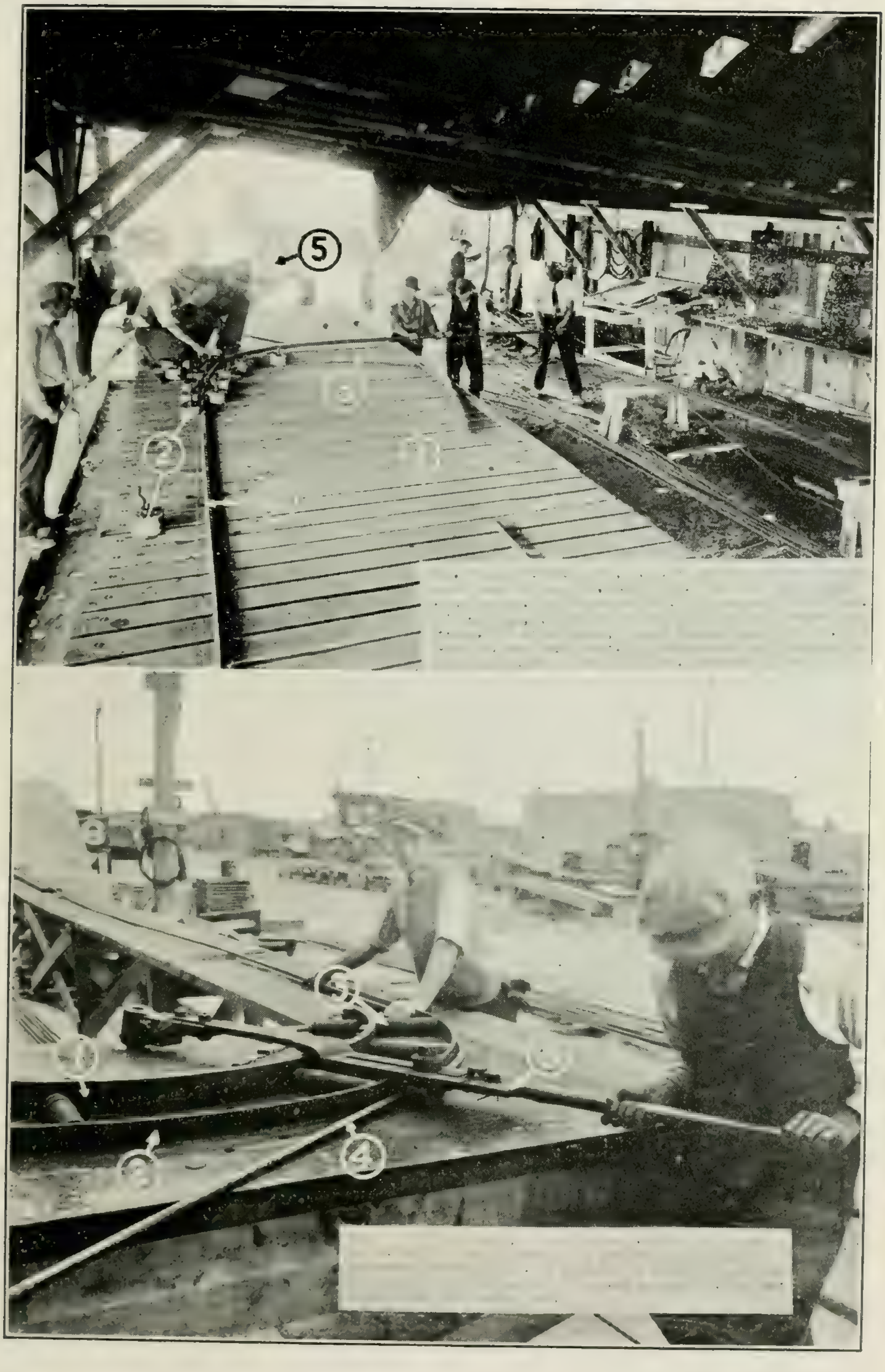


FIG. 34.—TABLES USED TO BEND FRAME RODS.

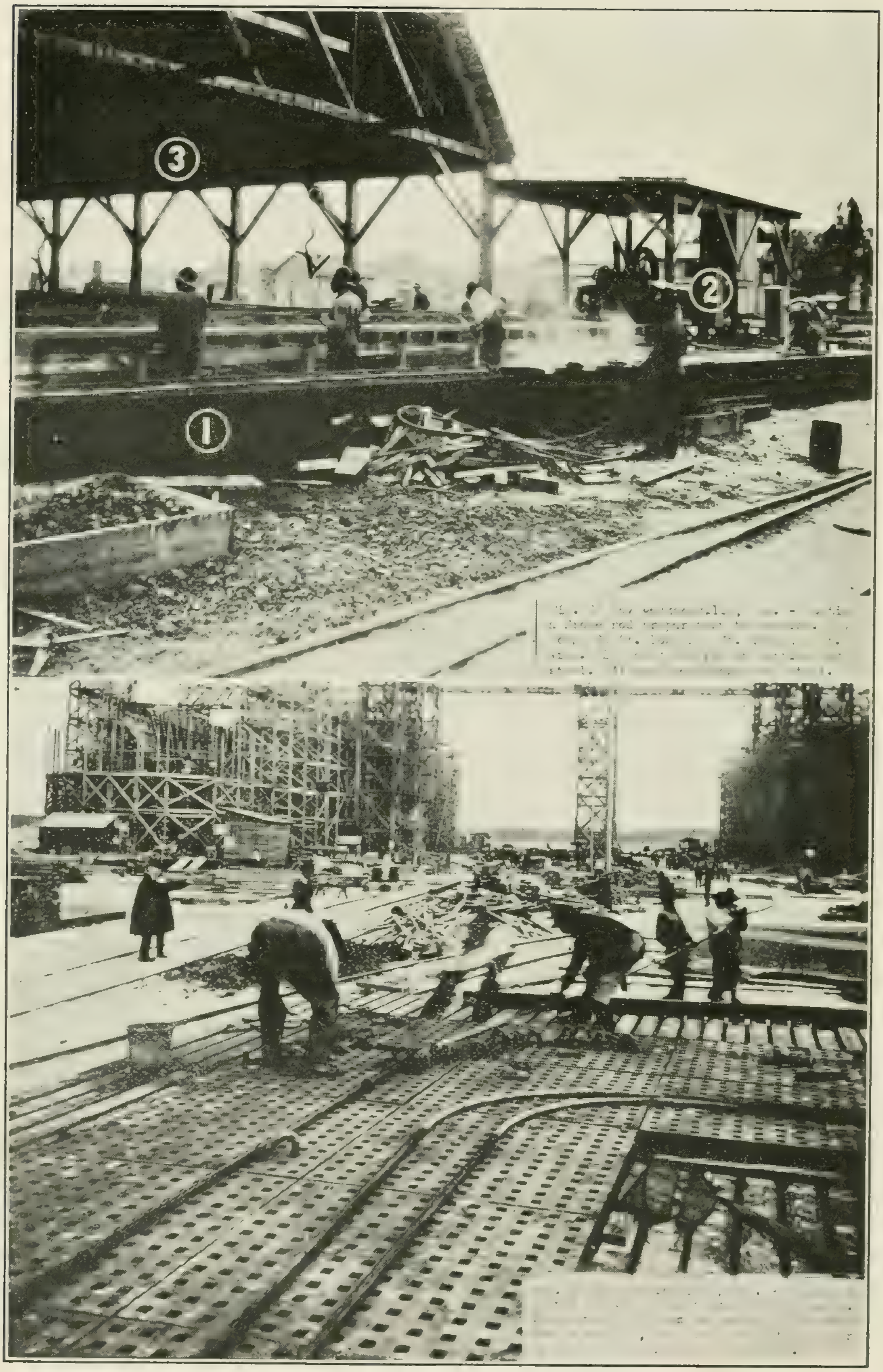


FIG. 35.—FORGE AND TABLE USED FOR THE HOT BENDING OF FRAME RODS.

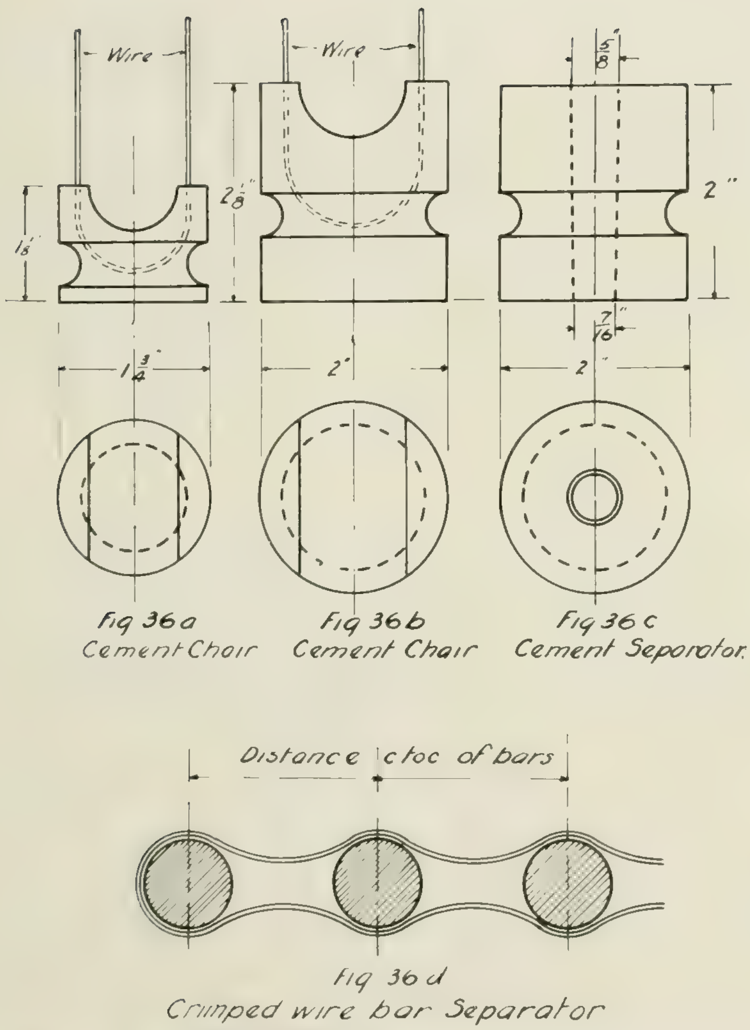
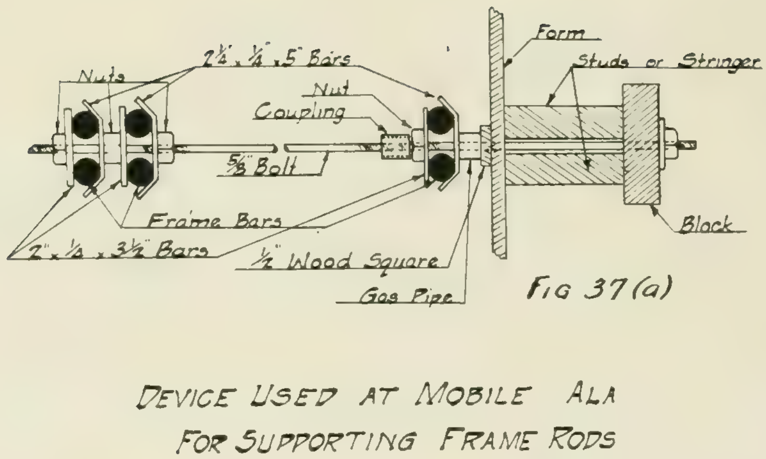


FIG. 36.—DEVICES USED FOR SUPPORTING AND PLACING REINFORCING STEEL.



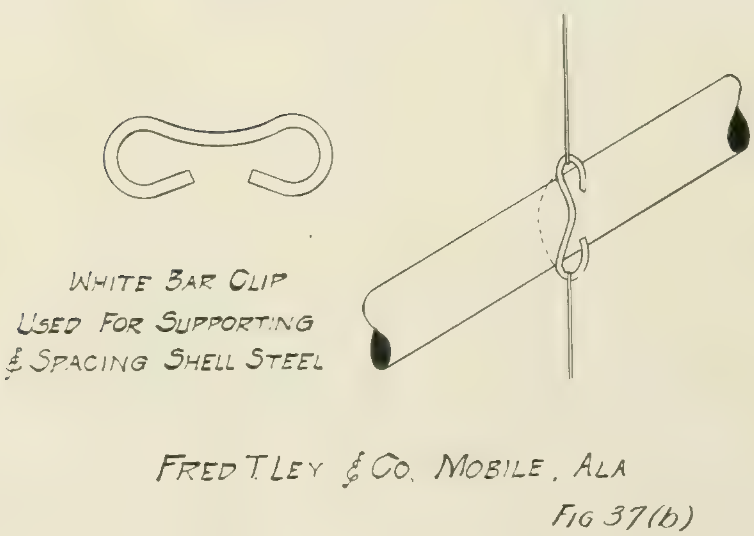


FIG. 37.—DEVICES USED FOR SUPPORTING AND PLACING REINFORCING STEEL.

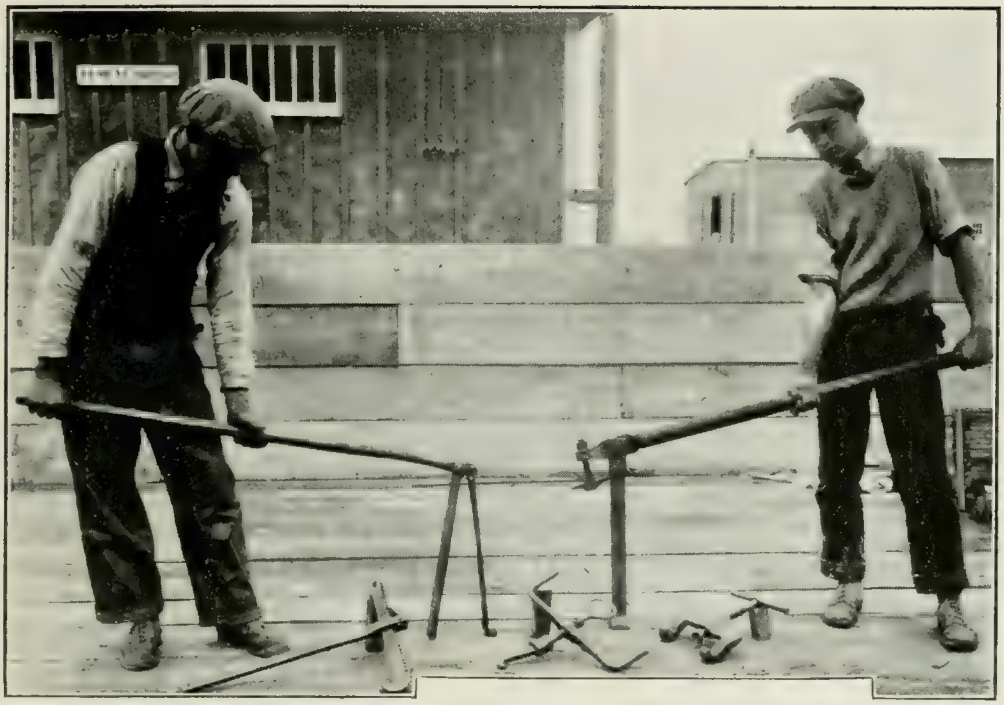


FIG. 38.—SPECIAL TOOLS USED IN PLACING REINFORCING STEEL.

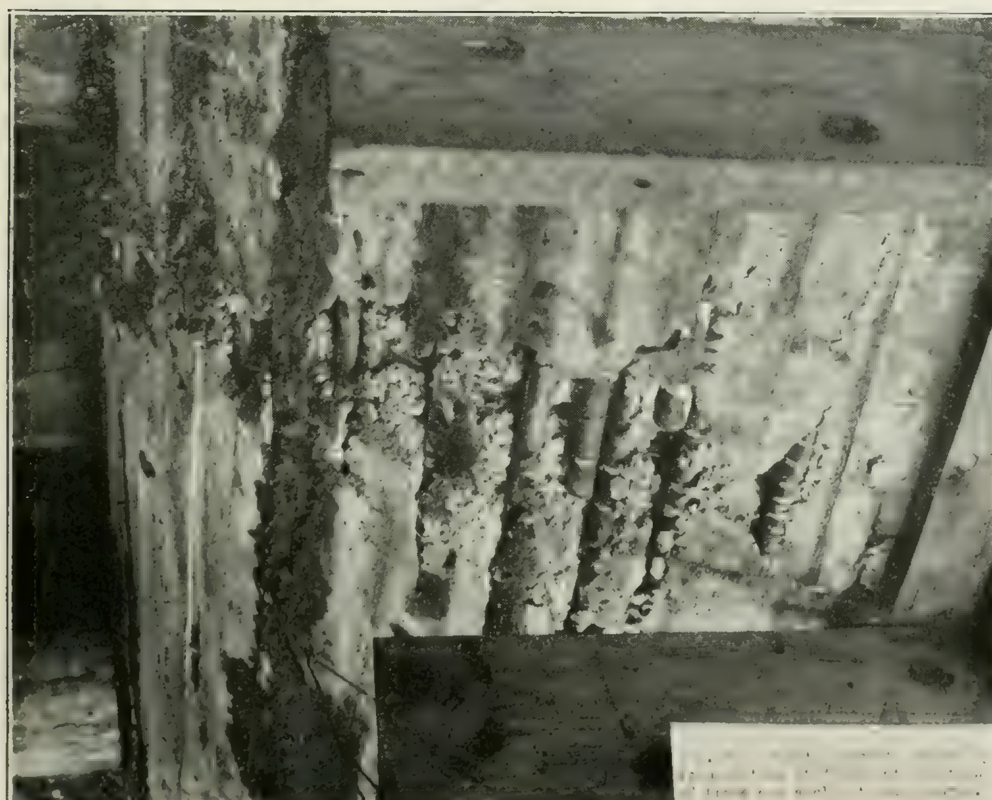


FIG. 39.—FAULTY CONCRETE CAUSED BY ACCUMULATION OF DEBRIS AT THE JUNCTION OF THE FRAME AND THE SHELL ON THE BRIDGE CURVE.

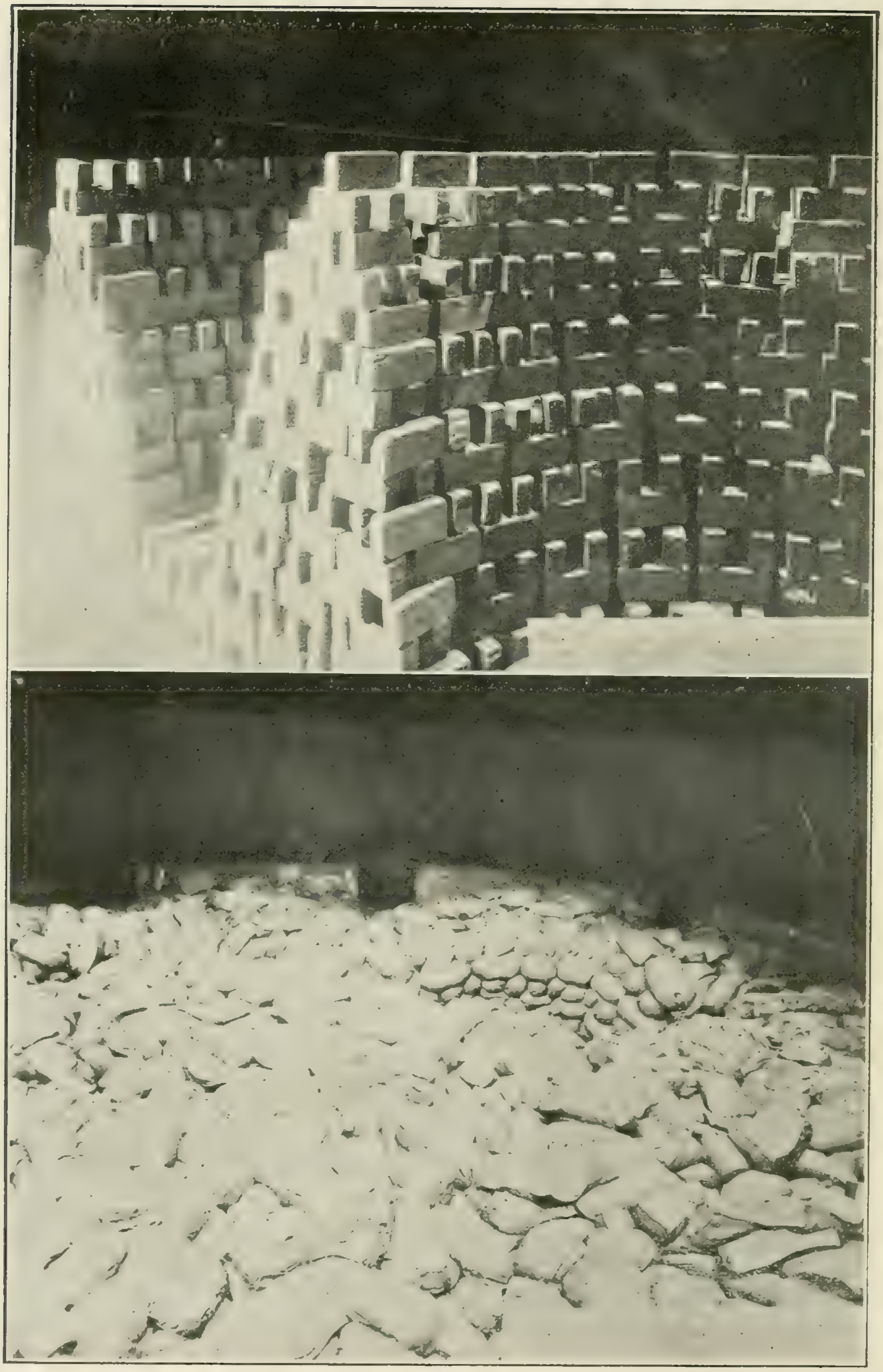


FIG. 40.—BRICK KILN METHOD OF BURNING LIGHT WEIGHT AGGREGATE.

the forms were erected and before the reinforcing steel was placed. Bolts, sockets, and similar fittings for securing equipment were usually set in fixed templets fastened in the forms Fig. 21 (a).

It was found that many of the castings such as bollards, winches, etc., were not drilled exactly according to plan and many of the anchor bolts would not meet up with the holes in the castings. It was found that the concrete could be readily drilled and new bolts set so this defect was not of great importance. It is preferable, however, to set the fittings on the forms with holding down bolts in sleeves attached to the fittings as shown in Fig. 21 (b) and pour the concrete around them, or another satisfactory method is to drill the base of fittings from the templets used for setting the bolts.

KIND AND QUALITY OF REINFORCING STEEL.

It was originally intended to use deformed square bars of the structural steel grade in sizes ranging from $1\frac{1}{4}$ to $\frac{3}{8}$ in. The square bar was selected with the belief that it could be concentrated more readily into a thin section than the round bar and a deformed bar was specified on account of its bond value. The square bars were found impractical because they assumed a twist or wind when bent to conform to the curves of the ship, thus requiring more space than the side dimension and the deformed feature was found objectionable from the construction standpoint because it was very difficult to weave the deformed bars on account of added friction due to the deformations. The bond stresses in the ship members were generally low and therefore a deformed bar was not necessary and it was abandoned in favor of the plain, round bar ranging in size from $1\frac{3}{8}$ to $\frac{3}{8}$ in. diameter, which has been used in all ships except the "Atlantus" and "Polias," the two experimental ships built by the Liberty Shipbuilding Co. and the Fougner Concrete Shipbuilding Co.

It was intended to use structural grade steel, but early in 1918 the War Industries Board ruled that because of the great demand for this grade of steel (largely caused by the steel ship program) no more structural grade steel reinforcing could be rolled in the East. It was necessary to substitute rods rolled from the discard croppings from shell ingots. This material, which may be classed as a hard steel, had an ultimate tensile strength of about 95,000 lb. per sq. in. and a yield point of about 60,000 lb. per sq. in. To provide for the use of this harder steel, two alternate specifications were written. The first, for the structural grade steel, is that recommended by the American Society for Testing Materials, Standard Specifications for Billet Steel, Concrete Reinforcement, Structural Grade. The second, for the shell discard material may be epitomized as follows:

1. The steel must be made from new billet, open hearth stock.
2. It must not contain more than .06% phosphorus.
3. Its physical properties must conform to the following minimum requirements:

Yield point......................	50,000 lb. per sq. in.
Ultimate strength................	80,000 lb. per sq. in.

$$\text{Elongation in } 8''\% = \frac{1{,}200{,}000}{\text{Ultimate strength.}}$$

Test specimens must bend cold, without cracking on the outside of the bent portion as follows:

Bars below $\frac{3}{4}$ in. in diameter must bend through 180° around a pin whose thickness equals the diameter of the bar. Bars $\frac{3}{4}$ in. in diameter or above must bend through 180° around a pin whose thickness is twice the diameter of the bar.

Typical results of tests of the steel are given in Table II. This hard steel caused considerable difficulty in the large sizes for it was very hard to bend to shape. Approximately 1,600 short tons of reinforcing steel was used in each of the 7,500-ton tank and cargo ships and 560 tons in the 3,500-ton cargo ship. The longest length obtainable was 60 feet.

HANDLING OF REINFORCING STEEL INTO HULL.

At all yards the shell and bottom and side frame steel was carried by hand labor from the steel yard or railroad car into the hull through openings left in one side of the outside forms near the quarter points of the length of the ship. The frame and side shell steel was hoisted by hand or block and tackle from the top of outside forms. At one yard, a temporary extension was made of the railroad track to the opening in the forms and the steel was dragged by hand labor from the car platform directly into the forms.

TABLE II.—TEST TO DETERMINE EFFECT ON STRENGTH OF THE REINFORCEMENT AFTER HEATING TO A CHERRY RED.

Portion of Bar Tested.	Yield Point.	Ultimate Strength.	Per cent Elongation in 8″	Per cent Reduction in Area.
Unheated portion	56,400	102,500	16.5	30.4
Junction of heated and unheated portion	58,600	105,900	11.6	30.4
Heated portion	56,400	102,500	15.3	34.6

Each figure is the average of two determinations.

NOTE.—The specifications call for a yield point of 50,000 and an ultimate of 80,000. It would appear from this table that heating to a cherry red did not change the tensile properties of the bars.

After the bottom concrete was poured it was necessary to close the openings in the lower part of the outside forms and the steel had to be hoisted over the top. At Oakland a power lumber hoist operating on the side of the staging was used to hoist the steel to a runway on the top deck, Fig. 19 (b), from which it was carried by hand onto the ship. At San Diego it was hoisted in bundles by tower revolving cranes and dropped into the ship. Mobile used a gantry crane; Jacksonville, derricks; and Wilmington, tower whirlers in the same manner.

FABRICATION OF REINFORCING STEEL.

At San Diego, all reinforcing steel was sheared to exact lengths and all bars including unbent bars were tagged according to detailed plans and sent to the forms ready to set into position, all joints and laps being fixed on the

plans. In all other yards, the unbent longitudinal and shell steel was sent to the forms without shearing or tagging, the staggering of joints and laps being provided for by instructions to the steel foreman. It is questionable whether the second method is equally as good as the first.

The bending of stirrups was originally done by hand methods in all yards. The bender consisted of an arm pivoted at one end and carrying a hardened roller which bent the stirrups around a fixed pin as commonly used in building work. One of these is shown in Fig. 33 (a). At Jacksonville, a unique power bender was devised and proved very satisfactory. It works in the same manner as the hand bender, except an air cylinder constructed of standard pipe and fittings with leather cup piston is provided to operate the bending arm, thus eliminating one man as well as considerably increasing the speed of bending. This device is shown in Fig. 33 (b) and is capable of bending two $\frac{5}{8}$ in. stirrups at a time.

The heavy frame bars, ranging from 1 to $1\frac{3}{8}$ in. diameter round, were bent by normal methods either on a hand bending table as shown in Fig. 34 or by a power bending machine such as the McKenna. Curved bends were difficult to make cold because the bars had a certain amount of spring and the templet or dogs around which the bending was done had to be set to allow for the recovery in the bar. The quality of bars was not uniform in this respect, which required testing each bar with the curved templet after bending. Only a fraction of an inch tolerance could be allowed for error in bending. At Jacksonville, Fla., all large bars with curved bends were bent hot upon a cast iron bending table about a steel templet previously set, and where angle bends occurred in the same bars they were heated and all bends completed at the same time. Bars for hot bending were heated to a dull red in a forge built as a plate metal box about 3 ft. wide, 3 ft. high and 60 ft. long, filled with sand to about 12 in. of the top, at which level a perforated pipe with removable $\frac{1}{8}$ in. pipe plugs was extended the entire length of the forge and supplied with compressed air at one end. A fire of coal and coke was built of any necessary length along the forge and accelerated by removing the pipe plugs through the range of distance required to be bent and at as many places along the 60 ft. bar as bends were required. The forge and cast iron table are shown in Fig. 35.

Hot bending proved very satisfactory as to accuracy, cost and speed, but considerable saving would have been made had two forges been provided for the one bending table. It was found impracticable to withdraw one heated bar from the fire and replace it with a cold bar to keep the heating process continuous without breaking up the fire and disturbing the heating of the other bars. The bending crew were not occupied in bending more than about one-third of the total time and could easily have bent the output of two forges whenever several bars were to be bent to one radius and resetting of the templets on the bending table was not necessary. That this process did not affect the strength of the bars is shown by the tests made at the Bureau of Standards, Table III.

The vertical shell steel, usually $\frac{5}{8}$ or $\frac{3}{4}$ in. diameter round, which has to conform where it is set into the bilge to the curvature of the ship, was bent

on a hand bending table if the curve was of large radius, or in power machines if of small radius or angle bends.

Where this relatively light steel of long lengths was bent at a distance from the point of construction, difficulty was experienced in some of the yards in transporting it to the forms and maintaining the original bend. At one yard, the bending table was placed within the hull forms and the vertical shell steel was carried directly from the bending table and set into position in the forms. If the other work to be done in the hull has been carefully planned so that the steel bending and erection gangs will not interfere with other labor units this method is quite satisfactory. It is preferable to have all possible work done outside the hull forms, for only a limited number of men can work efficiently within the hull.

TABLE III.—EFFECT OF FINENESS OF CEMENT ON THE STRENGTH OF CONCRETE.

Each figure is the average compressive strength of twenty 4 by 8-in. cylinders.
Atlas aggregate used: 1 volume fine to 2 volumes coarse; graded up to ½ in.; mix by volume; same brand of cement used.
Relative consistencies from 1.00 to 1.50, the gradation being the same in each case.

Mix.	Fineness (per cent Cement Passing a 200 Sieve).	Compressive Strength.	
		7 Days.	28 Days.
1:4	91.2	1,384	2,200
	84.2	1,170	1,816
1:3	91.2	2,040	3,030
	84.2	1,800	2,538
1:2*	91.2*	3,118*	4,248*
	84.2	2,642	3,770
1:1:½	91.2	3,540	4,455
	84.2	3,158	4,378
1:1	91.2	4,082	5,090
	84.2	4,014	4,848

Tests by Prof. Duff Abrams, Lewis Institute, Chicago, Ill.

* Mix used for concrete ships.

PLACING OF REINFORCING STEEL.

The exact method of placing the steel varied, depending partly upon the method employed in supporting the steel.

Following is a description of the method employed at San Diego for placing the shell steel.

On the inside of the outside forms the location of the outboard longitudinal bars were marked off starting at the keel. On these lines nails were partly driven into the forms in transverse rows about 8 feet centers indicating the location of the bars. The outboard steel is then placed directly in position on these nails as it is carried from the car.

It was more convenient for the workman walking back and forth if all the side shell steel is placed before placing the bottom steel. After placing the side, horizontal, outboard steel the vertical side shell bars are placed

followed by the diagonal spacing bars and the horizontal inboard shell steel as shown Fig. 7. The side shell steel is held away from the forms by means of small cement blocks wired to an occasional bar of the outboard layer of steel. The floor or bottom steel is then placed in a manner similar to the side shell steel excepting cylindrical cement blocks about 2 in. in diameter and of varying heights (see Fig. 36) are used to support the steel off of the forms. A cement block 2 in. high carrying the second or transverse layer of reinforcing steel was found to be preferable to a 1-in. block carrying the outboard steel as the latter would occasionally crush. Where the higher block was used the outboard layer of longitudinal shell steel was suspended to the transverse bars with No. 16 gage annealed wire ties attached to every alternate transverse bar.

Staging was erected on the inside of outside forms for the use of workmen in handling the side shell and frame steel. Three different methods were employed by the several contractors as shown in Figs. 5 and 16 (a). The method shown in Fig. 5 (b) and (c) is preferable in that it keeps a clear space for handling the steel at all elevations along the sides of the forms and requires no holes through the sheathing.

San Francisco used $\frac{3}{8}$ x $1\frac{3}{4}$-in. wood strips placed transversely at each frame (Fig. 7) lightly nailed to the inside of the outside forms for supporting the shell steel off the forms. Jacksonville used $\frac{3}{8}$-in. round bars in the same manner as the wood trips above but only across the bottom forms. The wood strips were removed after the concrete was placed and the surface was pointed up. The round rods were not removed, however. For the side shell steel, cubes of concrete were made as shown in Fig. 7 (b) with a groove in the top. This cube has a hole through the center perpendicular to the groove for bolting to the inside of the outside forms and the groove is so placed that the horizontal rod will be supported $\frac{3}{8}$ in. from the forms. The nut of the bolt is placed on the inside so after casting the concrete the bolt can be removed and the hole filled with cement. The cubes were spaced about 6 ft. longitudinally and 3 ft. vertically. The horizontal rods between the cubes were hung to the rods on the cubes by S-shaped wire hangers. As the vertical shell steel was placed the horizontal bars were wired to it and thus securely held in position. At Mobile, small cement or ceramic tile blocks were used to support the steel from the forms and a special prekinked wire was used to space and support the shell bars as shown in Fig. 36. The most satisfactory method was probably the one used at San Diego as described above. At Mobile, the shell steel was supported by the "White Bar Clip." A test panel, showing this system, is shown in Fig. 7 (d), and a sketch of the clip itself is given in Fig. 37 (b). The horizontal rods are hung from wires nailed to the forms Fig. 7 (d) and held in place by the clip.

Various means were employed for erecting the frame steel. Some form of clamp similar to that shown in Fig. 37 (a) was used in three of the yards. In some cases where the clamps were used the outboard frame bars were individually wired into position and the inboard frame bars were preassembled in the clamp as shown in Fig. 8 (a) and placed in position as a unit. At San Francisco the side frame bars were erected and then the bottom frame

bars. In one yard a temporary wooden horse was placed at the elevation of the bottom of the vertical side frame bars until they were wired into place with the stirrups. The horse was then removed and the bottom sections of frame bars were placed in a similar manner. At Wilmington, N. C., the frames were prefabricated in three sections one of which is shown in Fig. 8. The first unit consisted of the bottom or floor section up to the 6 ft. 4 in. water line and the other two sections extended from this line up to the deck line and over to the hatch girders or the center line of the ship. The prefabrication was done in an adjustable wooden frame which was set to a templet for each frame. No difficulty was experienced in setting these prefabricated frame sections. The bottom sections of the frames were picked up by their ends, which tended to shorten them slightly due to the sag, thus providing plenty of clearance to enter the forms. As soon as the frame was set in the floor on position and released, it sprung back into position, tight up against the shell steel. It was anticipated that there would be difficulty in setting the upper sections of the frames because of the lapping of the end bars of the two sections. Little difficulty, however, was experienced on this account.

Many special tools were designed to assist in erecting the steel. A number of these are shown in Fig. 38. No. 1 is a lever jack that was used for raising the bottom shell steel above the formwork for placing cement spacing blocks. No. 2 is a bar lever jack with a swivel tee used to force the frame bars up into the corners of the stirrups and to make the hooks of the stirrups fit closer around the shell steel. No. 3 is a ratchet lever used for drawing two bars together and is usually applied in tightening the two legs of a stirrup against the frame bars. No. 4 is a screw clamp for bringing the stirrup in tight to the frame bars at the shell and may be used in many cases where the ratchet shown in No. 3 can not be applied. No. 5 is a screw bar bender that is used for bending the stud bolts which support the inboard frame bars during construction.

The deck steel was placed in a manner similar to placing steel in floors in ordinary building construction. At Wilmington, the large longitudinal bars in the fillet of the deck was electrically butt welded by means of a type 10 A 22OV, 60 cycle, AC machine, Fig. 47, manufactured by the Federal Machine and Welder Co. of Warren, Ohio. This machine is equipped for the continuous welding of bars from $\frac{1}{2}$ in. to $1\frac{1}{4}$ in. in size either round or square. The bars come in lengths of sixty feet and the butt-welding machine was set on the deck about sixty feet from the aft end of the hull The bars were passed through the machine and over a series of iron rollers extending along the fillet. No difficulty was encountered in handling the bars or in placing them. Eight men could handle a 240 ft. $1\frac{1}{8}$ in. bar and place it in position in the fillet. Good welds were obtained in from 30 to 60 seconds. The hard steel was a little more difficult to weld than structural grade which was not obtainable at the time this work was done.

CLEANING OF FORMS FOR CONCRETING.

The cleaning of the forms preparatory to placing concrete is one of the very important matters in concrete ship construction. On account of the

relatively large amount of reinforcing steel in the structural members it is very difficult to remove sawdust, shavings and other debris which may collect at points of intersection of the steel such as where the keelsons, frames and bulkheads join the floor slab, and in the haunches in frames. If such debris has been permitted to accumulate at these points both water and air even under considerable pressure have been found to be ineffective for its removal. It is practically impossible by visual inspection to detect such accumulations after the steel is erected.

Large openings should be left in the outside forms under all keelsons, bulkheads and the like and at frequent intervals elsewhere over the bottom forms and constant vigilance should be maintained to keep the forms clean during the erection of the steel and inside forms. No unnecessary carpenter work should be done within the hull and such wood working as is done should be over a work box so that the waste will not fall into the forms. The clean-out holes shown in bottom forms of Figs. 5 and 7 were inadequate in size. It is particularly difficult to see the accumulation of debris under such masses of steel frames, bulkheads, etc. Fig. 39 shows the defective concrete caused by sawdust, etc., under a frame. The form was washed and supposedly cleaned and an inspection before pouring did not detect the dirt.

All forms were thoroughly wetted just before placing concrete and in some cases it was necessary to spray the reinforcing steel in order to reduce its temperature from exposure to the sun.

MATERIALS USED FOR CONCRETE.

A concrete was required which would have a compressive strength of not less than 4,000 lb. per square in. at 28 days and be of the lightest weight possible. While in the plastic state, it must be of such a consistency that it is possible to work it into place thoroughly imbedding the reinforcing steel and completely filling the forms.

In order to obtain the highest possible strength with a maxim quantity of aggregate, the specifications called for a special high grade portland cement which required that it be ground so that at least 90 per cent passed the 200 mesh sieve.

The value of this fine grinding is shown by the results of tests given in Table IV.

It was necessary to develop some new type of aggregate in order to reduce appreciably the unit weight of the concrete. After much investigation an aggregate was developed with which it was possible to decrease the weight of the concrete from 145 lb. per cu. ft. to 105–120 lb. per cu. ft. Since there are 2,800 cubic yards of concrete in each of the 7,500-ton ships, each pound reduction in the unit weight per cubic foot of the concrete represents an added carrying capacity of approximately 32.5 long tons in the ship. Thus a saving of 30 lb. per cu. ft. represented approximately 1,000 long tons additional carrying capacity in the ship.

The aggregate developed is a vesicular slag* made by burning suitable

* See Engineering News-Record issue, April 24, 1919, for complete description of product and method of manufacture.

clay in either brick or cement kilns to such a temperature that it bloats. After cooling it is crushed, screened and used as any normal aggregate. The fines passing a $\frac{1}{10}$ or $\frac{3}{16}$-in. screen are used as sand and the coarse from $\frac{1}{10}$ or $\frac{3}{16}$ to $\frac{1}{2}$ in. is used as coarse aggregate. In Fig. 40 are given views of the brick kiln method of manufacturing this material.

This aggregate being porous and very sharp was not as workable in the plastic concrete as might have been desired particularly if the fines lacked in content of dust. The gradation of the fines varied depending in part upon the type of crushing equipment employed.

In order to increase the workability of the plastic concrete in such cases, a very small quantity ($1\frac{1}{2}\%$ by weight or about 12% by volume of cement) of celite was employed. This material is a very light weight (about 12 lb. to the cubic foot) natural silica product of the following chemical composition:

Silica (SiO_2)	86.00*
Ferric oxide (Fe_2O_3)	1.61
Alumina (Al_2O_3)	2.99
Lime (CaO)	.44
Magnesia (MgO)	.72
Sulphuric anhydride (SO_3)	Trace
Ignition loss	7.90
Alkali by difference	.34
	100.00

The addition of this material also tended to prevent segregation of the coarser particles in the concrete.

The concrete mixture employed was usually composed of one part by volume of cement to two parts of total aggregate. The proportion of coarse and fine in the aggregate varied from $\frac{2}{3}$ part fine to $1\frac{1}{3}$ part coarse to 1 part fine to 1 part coarse. Table V gives a summary of some of the tests made from concrete used in the hulls.

EQUIPMENT USED TO TRANSPORT CONCRETE TO FORMS.

A different type of concreting plant was used at each yard. At Oakland, the plant consisted of two 24 cu. ft. Koehring mixers set in pits. The aggregate and cement was brought to the mixers in buggies. A 90-ft. elevator tower with one cubic yard automatic dumping buckets was set over each mixer. The buckets dumped into a hopper at the top of the tower. One double drum electrically operated hoist served both elevators. The forward tower distributed concrete through three main chutes which terminated in one yard distributing hoppers on the center line of the ship about 40 ft. apart. The after tower supplied three similarly located distributing hoppers in the after end. These distributing hoppers were arranged with four outlets as shown in Fig. 15 (b) and from them the concrete was passed through chutes to buggies and distributed by the buggies to the forms as shown in Fig. 14 (a).

* 16.76 is soluble in 10 per cent HCl and 11.24 is soluble in 5 per cent Na_2CO_3.

TABLE IV.—COMPRESSIVE STRENGTH OF CONCRETE MADE FROM THREE TYPES OF LIGHTWEIGHT AGGREGATE MANUFACTURED FOR THE CONCRETE SHIP SECTION OF THE EMERGENCY FLEET CORPORATION.

Manufacturers of Aggregate and Type of Kiln Used in Burning.	Brand of Cement.	Mix.	Consistency Drop in in.	Weight of Concrete, lb. per cu. ft.	Age When Tested (in Days).	Compressive Strength of Cylinders 6 in. diameter by 12 in. high.	Modulus of Elasticity.
Atlas Portland Cement Company, Hannibal, Mo. Burned in cement kiln.	Southern States Reground so 90 per cent passes a 200 sieve.	1 cement 1 agg. under $\frac{1}{10}$ in.—1 agg. between $\frac{1}{10}$ in. and $\frac{1}{2}$ in.	9.12	119.3	28	4,537	2,953,000
Los Angeles Pressed Brick Company. Burned in brick kiln (see Fig. 39).	Santa Cruz Reground.	1 cement $\frac{2}{3}$ agg. under $\frac{1}{10}$ in.—$1\frac{1}{3}$ agg. between $\frac{1}{10}$ and $\frac{1}{2}$ in.	9.19	105.2	28	3,861	......
		1 cement $\frac{2}{3}$ agg. under $\frac{1}{10}$ in.—$1\frac{1}{3}$ agg. between $\frac{1}{10}$ in. and $\frac{1}{2}$ in. celite.*	9.91	102.1	28	3,256	......
Copland-Inglis Company, Birmingham, Ala. Burned in brick kiln.	Giant Reground	1 cement $\frac{2}{3}$ agg. under $\frac{1}{10}$ in.—$1\frac{1}{3}$ agg. between $\frac{1}{10}$ in. and $\frac{1}{2}$ in.	8.50	115.6	33	3,576	2,800,000

* Diatomaceous earth: $1\frac{1}{2}$ per cent by weight of cement.

TABLE V.—TYPICAL STEEL ANALYSIS.

Steel Rolled by	For Use at	Physical Analysis.				Chemical Analysis.				Grade.
		Elastic Limit.	Ultimate Strength.	Per cent Elongation in 8 in.	Per cent Reduction in Area.	Carbon.	Manganese.	Phosphorus.	Sulphur.	
Atlantic Steel Company	Wilmington.	42,930	62,030	28.1	49.1	.18	.43	.042	.057	Structural
Llewellyn Iron Works	Oakland San Diego.	35,960	57,020	29.2	....	.20	.43	.018	.043	Structural
Tennessee Coal, Iron and Railroad Co....	Mobile....	62,230	97,356	16.9	34.2	.42 .52	.50 .90	.020	.03–.06	Shell discard
Bethlehem Steel Company	Jacksonville.	54,550	97,345	16.0	24.0	.50	.77	.035	.046	Shell discard

Since the hulls in this yard had been completely formed inside to the second deck before any concrete was poured a system of signals had to be installed in order to keep the men in the various compartments supplied with concrete as needed. For supporting the runway upon which the concrete for the top deck was transported in buggies a hanging scaffold was employed as shown in Fig. 20.

At Wilmington and Jacksonville, the concrete was transported from the mixers by bottom dump buckets swung on a whirler or derrick to a number of hoppers mounted on the staging at the top of the forms. See Fig. 10 (b). The concrete was conducted by gravity through pipes to mortar boxes from which it was shoveled or "pailed" to the forms. At Mobile, the concrete was elevated in a bottom dump bucket by a gantry crane and swung directly to a mortar box (Fig. 10 d) where it was deposited. From the mortar box the concrete was placed in the forms by shovels and pails.

The most unique equipment was that employed at San Diego. No. 24 motor-driven Koehring mixers located in pits discharge into Inslee controllable one-yard bottom dump buckets setting on push cars that are transferred from under the discharge chutes of the mixer to a position on the arc of a circle prescribed by the end of a revolving tower crane boom at a fixed slope. From this position the bucket is elevated and swung into position over the hull, lowered between the trusses where the concrete is dumped into hopper which travels along a runway on the center line of the ship Fig. 10 (c). The concrete is then discharged from the hopper at any point along the hull into chutes and delivered directly into forms or into flat boxes and then handled by shovels and coal buckets into the forms. There are four traveling hoppers which are operated on wooden tracks secured above the runway which is hung from the trusses as shown in Fig. 10 (c).

PLACING CONCRETE.

On account of the large amount of reinforcing steel and small clearances between the forms and steel it was found that the concrete could not be placed by the ordinary method of rodding or tamping unless a very watery concrete mixture of low strength was employed. As can be seen from an examination of the photographs much of the interior of frames, keelsons and shell could not be reached with a rod. The most thorough and practical means of settling the concrete into the forms and about the reinforcing steel was found to be by vibration. After some experimentation, it was found that small air hammers of commercial type (similar to Ingersoll-Rand "Little David" No. D) with blunt bitts from 12 to 36 in. in length held against the outside or inside of the forms (See Figs. 10, 14 and 41) near the point where the concrete was being placed would not only cause the concrete to flow, fill the forms and thoroughly embed the steel but would also increase the density of the concrete by driving out much of the entrapped air. Care was exercised not to let the hammer come in contact with the reinforcing steel. From 30 to 60 of these hammers were employed on each ship during concreting. For horizontal surfaces such as decks, a long shank bitt was placed in

the hammer and in most cases applied to the upper side of the forms through the concrete.

It was at first attempted to vibrate the outside forms by maintaining a hammer crew outside of the hull, but this method was found unsatisfactory for the hammers would at times through error be applied to surfaces where the concrete had been placed and partially set or they would not be applied where they could most effectively settle the concrete. It was found more satisfactory to use a long shank bitt and apply the hammer from the inside directly to the floor through the concrete as it was being placed and to the inside surface of the outside shell forms through the windows left in the inside forms or through small holes made in the inside forms which were later plugged.

The ordinary type of hammer while suitable for vibration of the forms

FIG. 41.—PNEUMATIC HAMMERS USED TO VIBRATE FORMS DURING CONCRETING.

requires the operator to absorb the recoil of the blow and in consequence it is necessary to relieve him at frequent intervals. At San Diego a special type of hammer was devised with an envelope casing which almost entirely absorbed the recoil. This hammer has proven to be very satisfactory.

There is a total of approximately 2,800 cu. yds. of concrete to be placed in the 7,500-ton ships, distributed approximately as follows: 600 cu. yds. in bottom to top of bilge, 1,200 yds. from top of bilge to and including the second deck, 800 yards from second deck to and including main deck and 200 yds. in superstructures. These various quantities were placed in one continuous operation at the rate of from 8 to 15 cu. yds. per hour.

With the large yardage of concrete to be placed in a continuous operation over a relatively large area in small units, it was necessary to organize the crews with considerable care and to plan a systematic operation. In some of the ships the placing of concrete was confined to two groups either starting

at opposite ends of the ship and working toward the middle or starting at the middle of the ship and working toward the ends. In most ships the depositing of concrete was carried on simultaneously in four sections of the hull, either two groups worked from the ends toward the middle and another two worked from the middle toward the ends, or two groups each started from the third points in the length of the ship and worked simultaneously toward the middle and ends of the ship.

There is a difference of opinion among the contractors as to whether the concrete should be placed in the bottom floor and allowed to flow under the bottom frame forms before filling the frame forms, or whether the bottom frame forms should be first filled, the concrete being permitted to flow out underneath into the floor before filling the floor forms. Defects have been found in some of the concrete placed by both methods. The results obtained are apparently dependent upon the care exercised.

Since the concrete was deposited continuously over periods of from 50 to 100 hours, the men worked in two or three shifts of 8 to 12 hours each. Great care and attention must be given to the placing of each shovelful of concrete into forms or defects of a serious nature will result. All the workmen connected with the placing of concrete should be schooled and instructed with regard to their individual responsibilities for inspection alone cannot insure a good job and one careless workman in this position can cause major defects.

No difficulty was experienced in making water-tight construction joints between the several pours of concrete. The surface where new concrete was to be joined to old was thoroughly roughened by chipping with the pneumatic hammers and any soft film of cement which might have formed was removed. The surface was then cleaned with compressed air or water and thoroughly saturated with water just before the placing of concrete of the next lift was started. In some cases, the surface of the hardened concrete at the joint was grouted also, but this was not necessary with the very rich mixture employed if care were exercised to insure that the first batches of concrete were thoroughly worked into the surface.

PATCHING AND POINTING THE CONCRETE.

Defects of at least a minor character have been found in the concrete of all hulls. This has necessitated some patching and pointing.

In most cases the patching was done by hand. The cavities were well cleaned, all loose material and dust removed and the old concrete thoroughly saturated with water. A mortar of usually one part cement to two parts of sand was mixed with water to a moist earthy consistency; that is, it was just wet enough so that when formed into a ball in the hands it could be made to cling together. This mixture was then pounded into the cavity with a hand hammer and hard wood stick or block. Excellent results were obtained by this method and practically no leaks have occurred around patches.

In some cases, holes were found which extended entirely through the shell or the defects extended to such a depth that it was necessary to cut away the concrete through the shell. In these cases, the repairs were

made either by placing forms on both sides, securing them into position by bolting through and placing the concrete as originally or by placing a form on one side only and depositing the concrete as described above for patching.

Where care has been exercised, the patching has been uniformly satisfactory and water-tight walls have been secured.

The inspection must be very thorough in order to uncover all defects, for in some cases the surface may appear sound and uniform or show only a slight cavity, but by tapping it will be found the cavity extends for some distance into the mass.

The cement gun has been used successfully in some cases for patching but satisfactory results are obtained by the hand methods if carefully done.

LAUNCHING.

All of the concrete ships described in this paper with the exception of the two small ships built in private yards were built for sideways launching.* The transferring of the hull from the ways to the water is accomplished in the following manner:

Alternate sections of the blocking and forms which supported the hull during construction are removed and a temporary cribbing is inserted and brought to bear by driving in wedges. This temporary blocking is shown in Fig. 24 (b) and 24 (c). As the forms and original blocking are removed the concrete is pointed, patched where necessary and painted.

The remaining sections of form blocking are removed and the patching and pointing of concrete is completed. The launching ways are set in place. (See Fig. 24.) The launching packing is then placed, the temporary blocking removed, transferring the load to the ways through the packing and the ship is launched.

It is obvious that the details of the first two operations will vary with the method used to support the bottom forms. As was outlined in the section dealing with blocking, scaffolding, etc., the forms at San Francisco were supported on prefabricated trusses running transversely across the ship. These trusses were spliced at the centerline and tapered from the bilges down to the keel. After concreting, the proper lines of trusses were pulled out from under the ship, allowing the forms to be removed. (Fig. 4 a.) A similar method was used at Jacksonville and Wilmington. Fig. 4 (b) shows clearly the construction of the trusses.

At San Diego blocking was used instead of the trusses. In Fig. 42 the three stages of removing this blocking and the bottom forms are shown. It will be noted that two of every three panels (each panel is 8 ft. 6 in.) were removed and the load carried on the third. The exposed surface was painted, cribbing installed and finally the last section of forms removed.

At Mobile a combination of blocking and struts was used to carry the bottom forms Fig. 4 (d). Immediately before launching the weight of the ship is carried to the foundations through the temporary blocking and through

* The reasons which led to the adoption of side launching are outlined in a paper by A. L. Bush, "Layout and Equipment of the Government Concrete Shipyards," Pro. Am. Concrete Institute, 1919, this volume, p. 215.

a series of wedge blocks placed under the keel. The launching packing is in place on the greased ways but does not as yet carry any load. The first operation of launching is to drive in wedges set in the packing so that the packing will bear against the ship and thus take some of the hull's weight. This may be done in one operation, but the general practice is to set the wedges up in two or more rallies. The temporary wedges and keel blocks are then driven out with battering rams, leaving the entire load on the packing. The blocking is usually made up with a block wedge held together by a steel strap and pin and with its sloping faces heavily greased. When the pin is driven out with a sledge the block may usually be knocked apart with the ram.

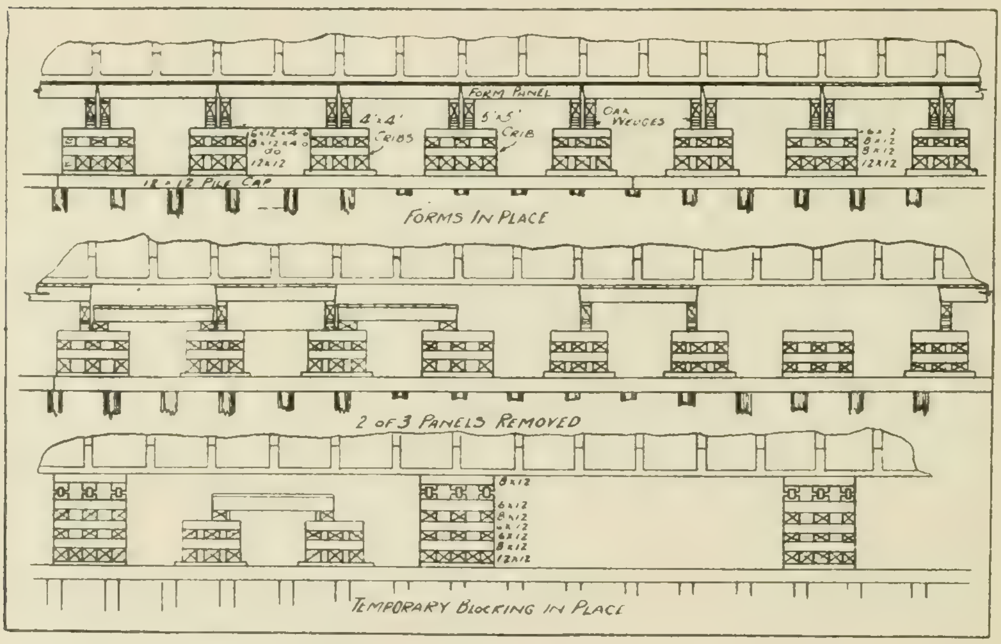


FIG. 42.—METHOD OF REMOVING BOTTOM FORMS AND INSERTING TEMPORARY BLOCKING USED AT SAN DIEGO.

Provision is always made, however, to split it out with steel wedges in case it jams.

With the blocking out, the ship is restrained from sliding by a series of so-called "daggers" and "triggers" at the bow and stern. Fig. 24 (a) shows a typical arrangement of these members. The thrust of the ship is taken into five daggers (Item 1) each of which bears on a trigger (Item 2). The thrust is carried across the triggers chiefly to a bolster block bolted to the side of the way and thoroughly braced by either one or more spins. The dagger is set so that the produced line of the face nearer the way clears the bolster by about an inch. The trigger is prevented from kicking out by a heavy line (Item 4) which passes under the ship and is anchored to a deadman on the inboard side of the building way. Each of these slings is carried over a chopping block (Fig. 25) at which a man is stationed with a broad axe. After

all blocking has been removed the launching master, standing amidships on the inboard side, gives a signal and all the lines are cut. The trigger is kicked out by the dagger, which falls clear, and the hull slides down the ways (Fig. 26).

The system just described was the one used at Mobile. As is shown in the figure there were five triggers, both at the bow and stern. Under the remainder of the ship there were thirty-five launching ways 8 ft. 6 in. on centers, each coming under a frame. The packing was built up to extend across three ways and there were four sets to a transverse section of the ship exclusive of the keel packing which is individual for each way. These sets do not line up transversely but each successive one starts one way further astern so that one transverse set of packing rests on six different ways. For example, numbering the ways back from the bow, the first complete outboard packing rests on ways 7, 8 and 9. The set just inside it is on ways 8, 9 and 10; the one inside that on 9, 10 and 11; and the inboard set on 10, 11 and 12. This staggered arrangement was used so that there would be a greater certainty of all sets of packing starting to move at the same time.

At San Francisco the details differed materially from these used at Mobile. Instead of the conventional dagger and trigger release, they used a cradle both fore and aft. After the load had been taken on, the packing lashings from these cradles to deadmen restrained the tendency of the hull to slide. These cradles are shown in Fig. 24 (c). It is also to be noted that the packing, some of which can be seen in the same photograph, extends across two ways instead of three as was the case at Mobile.

This packing was built up in five transverse sets, one under the center keelson, two under the bilges and two under the longitudinal bulkheads. The sets were not staggered, but two pieces of 2 x 8 ran transversely connecting the five units of each set.

At the date of writing, no 7,500-ton tankers have been launched at any yards other than Mobile and Oakland. At Jacksonville and San Diego, the method used at Mobile, with possible slight variations, will be followed.

The chief difference in the two methods described is in the releasing device. Mobile, with the dagger trigger method, followed very closely the conventional side launching procedure used almost exclusively on the Great Lakes and to some extent along the Atlantic seaboard. This method is just as positive and probably a little safer than the cradle scheme used at Oakland. In the latter it is necessary to use heavy timbers in fairly long lengths rigidly fastened together. Experiments conducted by the Concrete Ship Section on the sideways launching of steel ships show that velocity of twenty-three feet a second is attained. It is obvious that if any of the cradle timbers should work loose and jam that the force of a 5,000-ton hull moving at this speed would be more than ample to punch the timber through the shell. In the trigger arrangement no such long sticks are present. This actually happened on the side launching of a concrete barge when a cradle timber broke, jammed in the mud just forward of the launching ways and ruptured the shell of the vessel.

Some of the dimensions and figures of the Mobile launching system follow:

Slope of launching ways, $1\frac{1}{4}$ in. to the foot.
Slope of trigger ways, $1\frac{1}{4}$ in. to the foot.
5 triggers forward and five aft.
35 launching ways—all ways spaced 8 ft. 6 in. on centers.
Launching ways, 18 x 30 in. hard pine.
Packing (mostly), 12 x 12 in. hard pine.
Bearing stress on grease, $2\frac{1}{4}$ tons per sq. ft.
Trigger rope, 1 in. diameter Manila hemp.
Launching grease.
$\frac{1}{16}$ in. stearine on slides and ground ways.
$\frac{3}{16}$ in. launching grease on slides and ground ways.
$\frac{3}{8}$ in. launching grease between packing and ways.

At the date of writing, the Fleet Corporation has launched three 7,500 D. W. T. concrete tankers and one 3,500 D. W. T. cargo ship by the side launching method described. In addition, one each experimental 3,000 and 3,500 D. W. T. cargo ships have been launched endways. All six launchings have been thoroughly successful and without accident or damage to the hulls.

PAINTING.

Both the exterior and interior of all ships are painted.

The disintegration of reinforced concrete in sea water is due to the penetration of the water into the pores of the concrete in the submerged portion of the concrete, its absorption by capillarity into the air exposed portion and the evaporation of the water with a resulting concentration of sea salts causing an accelerated corrosion of the embedded reinforcement which splits and spalls the concrete.

The effectiveness of the protection to the steel provided by the rich concrete mixture used in ship construction is unknown.

As a precautionary measure, it was deemed wise to apply a paint coating to the exterior below the water line to reduce penetration and above the water line and on the interior surfaces to reduce evaporation of any water which may be absorbed by capillary action. In the tank ships it was also deemed advisable to apply an impermeable semi-elastic oil proof coating to the inside surface of all oil compartments. The bottom of a concrete ship will foul to the same extent as a steel ship unless it is protected; therefore, an antifouling paint was also applied.

Many tests have been made and a large number of different paint combinations have been tried but experience has not been sufficiently extensive to fully demonstrate their value. It would appear that the bituminous paints are the more satisfactory where continuously exposed to water. Following is a brief description of the various combinations now being used on the different surfaces.

All paints except the bituminous coatings were applied by air brushes as shown in Fig. 23.

It was originally intended to apply two priming coats of a $7\frac{1}{2}$ per cent

solution of magnesium fluosilicate over all surfaces of the hull. The purpose of these coats was to neutralize the alkali and harden the surface. This treatment was necessary where it was intended to apply an oil or varnish paint. It was found, however, that the varnish paints were of uncertain durability on the exterior surface of the hull and a bituminous paint was substituted which obviated the necessity for the application of the magnesium fluosilicate solution. In the case of one hull a rapid oxidizing vegetable oil known as "Repello" was substituted for the magnesium fluosilicate. This material not only neutralizes the alkali but considerably reduces the absorption of the concrete.

The following combination of coatings have been applied to the outside surface below the light draft line. Either two coats of magnesium fluosilicate, two coats of spar varnish and one coat of antifouling paint or three coats of bituminous paint and one coat of McInnes antifouling paint.

The outside surface above the light draft line is covered with the same coatings as the surface below the light draft line, except the antifouling coat is omitted and where the two coats of spar varnish has been applied it is followed by a third coat of gray enamel.

The weather deck is covered with an asphalt mastic one-quarter to three-eighths inch thick which acts as a wearing surface and will prevent leakage through cracks if any occur.

The interior surfaces of all oil compartments are covered with two coats of magnesium fluosilicate and three coats of spar varnish. In some cases a gray enamel is substituted for the third coat of varnish. Cheese-cloth is placed as reinforcement between the layers of varnish on the oil-exposed surface of all bulkheads which are exposed to oil on one side only. Cheese-cloth is also placed in a similar manner on the inside surface of the shell of the hull frames in the oil compartments.

The interior of all water tanks are covered with either three or four coats of bituminous paint.

The interior of cargo holds and boiler and engine room compartment are coated with either one or two coats of magnesium fluosilicate and three coats of spar varnish; or two coats of magnesium fluosilicate, two coats of spar varnish and one coat of gray enamel; or three coats of bitumen.

OUTFITTING.

After the concrete hull has been launched, the task of installing the machinery and piping, erecting the cargo handling apparatus and rigging and fitting up the accommodations for the officers and crew still remains before the ship is ready for sea. These items are all included under the heading of "Outfitting."

At the yards in Mobile, Oakland and San Diego, the superintendent who constructed the hull for the corporation is also doing the outfitting and from these yards the ships are to be turned over ready for service. The ships built at Wilmington and Jacksonville are surrendered after launching to the Jacksonville Ship Outfitting Co. yard at Jacksonville, Fla.

The first of the three items mentioned, the installation of machinery, is the most important part of the outfitting. Besides the main engine and boilers, it includes all the auxiliaries such as pumps, condensors, steering gear, refrigerating machinery, generators, capstans, windlass, and cargo winches.

In the 7,500 D. W. T. ships, both cargo and tankers, the propelling equipment is the same. The engine is triple expansion, three cylinder, vertical inverted direct acting Stephenson link type, with cylinders $24\frac{1}{2}$ in., $41\frac{1}{2}$ in., and 72 in. in diameter with a 48-in. stroke and is to develop 2,800 hp. at 88 rpm. (corresponding to a cross head speed of 700 ft. per min.). It is to operate at this speed with 200 lb. steam pressure. Steam for this engine is generated in a battery of three oil burning Foster Water Tube boilers, which are located in a boiler room just forward of the engine room. Each has an external heating surface of 3,050 sq. ft. and is to be built for a working pressure of 225 lb. per sq. in.

Both engine and boilers and all the rest of the outfitting equipment is practically the same as that used on a steel ship of the same size. The only points of difference are in the details used to fasten the various pieces of equipment to the hull. The heavier machinery and the boilers are supported on steel grillage which is fastened to the frames by means of bolts set in the concrete. (See Fig. 11.)

At Jacksonville, the steel grillage was omitted under the engine and a concrete foundation was provided to which the engine bed was directly secured by bolts passing through pipe sleeves cast into the concrete.

The location of these inserts is determined by means of templets on which the bolt holes are spotted either from the machinery drawings, or better still, from the base of the machinery itself. An example of how such templets are used is shown in Fig. 21 (b) in which Item 1 is a templet to locate the bolts for the foundation of the flying bridge. In this case and for some of the other lighter members such as ladders, etc., the pipe sleeves are sometimes omitted and a bolt with a large washer is placed directly in the concrete. The deck machinery subject to much vibration such as cargo winches, windlass and capstan are set on 2-in. oak bolsters. In the case of some of the smaller equipment such as mooring bits, the part is placed in position on the forms before concrete is poured. (Fig. 21 (b), Item 2.)

It has been found difficult to place the smaller bolts and inserts so that they would fit the equipment unless they are set with templets taken directly from the casting. The drilling of ship castings and fittings is usually not exact according to drawings. Very little difficulty has been caused on this score, however, for if a bolt does not line up with the hole in the fitting a new hole is easily drilled in the concrete and a new bolt grouted in place.

The cargo handling equipment of the 7,500-ton cargo ships is similar to that on a steel ship of the same size. There are two masts (See Fig. 3) each with four 5-ton cargo booms and each serving two hatches. In addition, one 30-ton boom can be rigged to forward mast if necessary. When this is used it is necessary to shore the decks under the boom and to install additional guys as the regular shrouds will not carry the load.

On the tankers no such elaborate equipment is necessary. Here most of the cargo is handled by pumps located in special pump rooms adjacent to the main engine room. The installation of the oil piping is similar to the installation in a steel ship of similar type. In Fig. 2 the holds for the oil cargo are marked "Cargo Oil." Immediately forward and aft of these compartments are small holds for dry cargo. To serve these holds each of the two masts is provided with one 5-ton boom.

There is very little to be said in regard to the fitting out of the crews' quarters, for they differ in no way from the quarters on any of the other Emergency Fleet ships. The exterior walls of the deck erections on all the ships are of concrete except the poops on the 7,500-ton D. W. ships, which are of wood bolted to the concrete by through bolts. None of the crew are quartered in the forecastle. The eight seamen have two large staterooms in the poop and the rest of the ship's company are quartered in the bridge house.

TRIAL TRIPS.

After the outfitting and installation of machinery has been completed, it is customary to subject the ship to two tests before it is accepted as ready for service. The first test is made while the ship is tied at the wharf and is known as the "Dock Trial." At this time, the boilers are fired, and the engines run at full speed for a time long enough to satisfy the trial board as to their proper performance, and the auxiliary engines are also tested.

After the dock test, the defects discovered are remedied and the second and final test made. In this test the ship is taken to sea, unloaded, and made to perform all the evolutions to be encountered in actual service.

At the time of writing, only the 3,000 and 3,500-ton D. W. experimental concrete ships have made their trial trips.

On the trial trip of the "Polias" (3,500-ton D. W.) the log shows that she was run for six hours at full speed, averaging 10.5 nautical miles per hour. Over a seven-mile stretch of this course, the speed was figured as 11.4 nautical miles per hour. At this speed her engines were making 93 rpm. and indicated 1,364½ hp. This part of the trial is run to test a ship's propulsive equipment and determine if she makes her guaranteed speed which in the case of the "Polias" is 10.5 nautical miles per hour.

After the speed run had been made, the maneuvering powers were tested While the ship was running at full speed she was steered through a figure 8. The log shows that the complete turn on the port wheel was made in 6 min. 25 sec., and on the starboard wheel in 6 min. The wheel was thrown from hard over to hard over in 12 sec., which is a very good average for a ship of this size.

Next the auxiliaries were tested. The dynamo was loaded to its maximum, the ice machine tried out, feed pumps tested and the temperature of the feed water and stack gases taken. Both anchors were let go and hove up in fifteen minutes, and the hand steering gear used to steer the ship; finally, the engines are reversed and the ship run full speed astern for ten minutes. With

the ship going full speed ahead, reversing the engines brought her to a standstill in three minutes.

These are the standard tests made on the trial trip of any ship. Both the "Polias" and the "Atlantus" handled very much as a steel ship of similar size and lines and with the same type engines would handle. The chief difference which was noticed by everyone on the trial boards was the marked lack of vibration on the concrete ship. One other point, which was found in both concrete ships, is that they backed nearly straight and can be steered slightly while backing. A steel ship with a right handed engine always swings her stern to port when backing, and usually does not answer to her helm. Why the concrete ship should show this difference is not known, and it may not hold for the other ships, but it is a point common to both of the Emergency Fleet Corporation's concrete ships which have so far been tested.

COST AND PROGRESS RECORDS.

Weekly progress and cost reports were received from all concrete shipyards. A card of accounts (Fig. 47) was prepared upon which the cost was reported segregated in fifteen main items. The report of progress and cost includes a statement of the direct labor charge on each of these items, the man hours of work done on each item for the week covered by the report and the quantity of work done on each item. In addition to the fifteen items of hull construction, report is also made on the distributed prehandling expenses of chief materials entering into the construction of the hull. The setting of propellers, line shaft and rudders is also covered by this report, for although this work is really part of the outfitting, a large proportion of it may be done while the hull is still on the ways.

Each item of construction has been analyzed on the basis of total man hours necessary for its completion and a comparative value set on each, so that the total of the items adds to 100. If the figures showing the percent of each item erected is multiplied by this so-called "unit value" and the product of these terms are added, the sum will be the percentage complete of the entire hull.

TOTAL AND UNIT COST.

At the time of writing, total costs are available on the hulls of four 7,500 D. W. T. reinforced concrete oil tankers. The net cost for the hull alone ranges from about $600,000 to $700,000. The relative distribution of this cost for hull 1,715, 7,500-ton tanker as built by the Fred T. Ley & Co., Mobile, Ala., is shown on Fig. 43, no deduction being made for salvage value of materials.

As yet none of the larger ships have been completely outfitted and no figures are now available as to the final total cost of this work. The partial figures now on hand would indicate that the outfitting will cost, including material and labor, about $600,000 for each 7,500-ton ship. The tankers will therefore cost in the neighborhood of $1,250,000, or about $167 per D. W. T. This figure should be compared with the $225 to $300 per ton cost for steel tankers built during the same period.

A detail statement of the unit cost of the 15-hull construction items is given on Table VI to Table XX. It must be remembered that the ships being built at Wilmington are of a different type and present more difficult construction than do the tankers. Their lines are the conventional ship lines unlike the simplified lines of the tankers, in which plane sections have been used wherever possible instead of the complicated molded surfaces of the Wilmington ships.

In Fig. 44 is given a time progress curve for the different items in the construction of the 7,500-ton tanker, hull 1,663 constructed by the San Francisco Shipbuilding Co. at Oakland, Cal. The total time required for

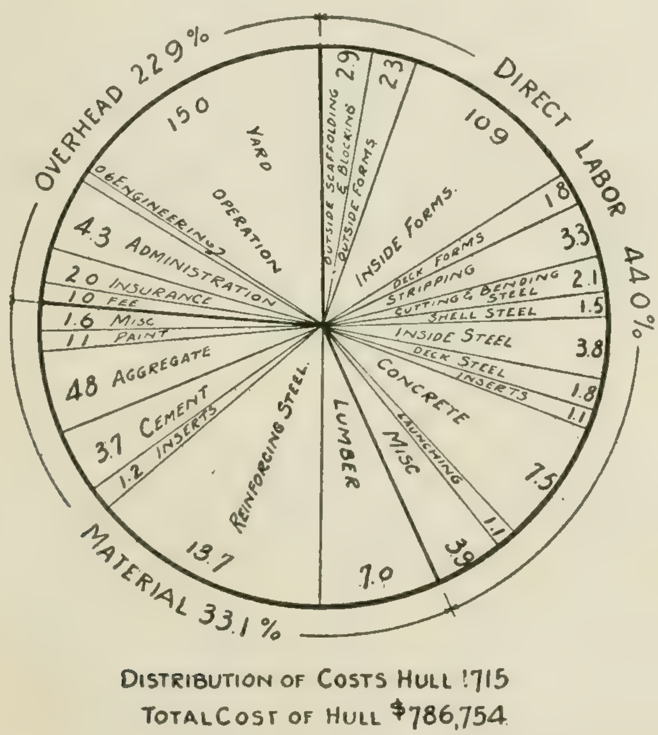


FIG. 43.—DISTRIBUTION OF THE COST OF HULL 1715.

this ship was the shortest of any of the larger hulls so far built but the relative proportion of the total time consumed on each item, as well as the time of starting and completing each item and the arrangement with each other, is probably typical. A time progress curve showing comparative rate of progress on each of the ships being built in the Government Agency concrete shipyards is shown in Fig. 45. It was originally intended that construction should proceed much more rapidly than is here indicated, and when work started on the earlier hulls the progress was greater. This extra speed necessitated a great deal of overtime work, and costs were sacrificed to speed. With the signing of the armistice the policy of the Emergency Fleet Corporation was changed, and any work being done at high cost in order

TABLE VI.—OUTSIDE SCAFFOLDING, BLOCKING AND ROOF TRUSSES—LABOR COSTS.

	Date of Figures.	Hull No.	Quality of Lumber in M. ft. B. M. for One Hull.	Per cent of Item Complete.	Total Cost of Item to Date.	Unit Cost of Item, M. ft. B. M.	Total Man Hours on Items to Date.	Organization of Crews Working on this Item.	Approximate per cent of Each Trade Used on Item.	Average Hourly Wage of Trades.
Jacksonville....	8/27/19	1707	581,133	100	$35,801.82	61.61	40,765¾	Carpenters	95	$.80
	8/27/19	1708	570,954	85	34,380.78	62.00	46,218¾	Laborers	5	.46
Mobile........	9/3/19	1715	560,000	100	$23,400.29	41.79	33,629½	Carpenters	55	$.80
	9/3/19	1716	560,000	100	23,767.38	42.44	34,320½	Bolters up	25	.58
								Laborers	20	.46
San Diego.....	8/20/19	1723	588,000	100	$17,957.05	30.54	24,728½	....	......	....
	8/20/19	1724	588,000	100	16,859.55	28.60	24,395½	....	......	....
San Francisco..	8/24/19	1662	466,000	100	$26,604.97	57.10	32,342½	....	......	....
	8/24/19	1663	466,000	100	15,534.62	33.34	21,123	....	......	....
Wilmington....	8/27/19	1560	201,400	100	$22,444.26	112.00	....	....	......	....
	9/17/19	1562	173,900	100	10,734.41	61.73	....	....	......	....

TABLE VII.—OUTSIDE FORMS—LABOR COSTS

	Date of Figures.	Hull No.	Sq. ft. of Contact Surface.	Per cent of Item Complete.	Total Cost of Item to Date.	Unit Cost of Item per sq. ft.	Total Man Hours on Items to Date.	Organization of Crews Working on this Item.	Approximate per cent of Each Trade Used on Item.	Average Hourly Wage of Trades.
Jacksonville....	8/27/19	1707	51,724	100	$7,732.50	.15	9,684½	Carpenters	95	$.80
	8/27/19	1708	51,724	100	8,255.56	.16	11,120	Laborers	5	.46
Mobile........	9/3/19	1715	54,000	100	$18,430.26	.34	27,256	Carpenters	80	$.80
	9/3/19	1716	54,000	100	16,094.19	.30	24,244	Laborers	20	.46
San Diego.....	8/20/19	1723	51,000	100	$15,020.00	.294	19,532½	....	......	....
	8/20/19	1724	51,000	100	13,701.50	.268	17,620	....	......	....
San Francisco..	8/24/19	1662	50,000	100	$18,576.34	.372	24,542½	....	......	....
	8/24/19	1663	50,000	100	11,531.04	.231	14,929½	....	......	....
Wilmington....	8/27/19	1560	27,000	100	$15,500.50	.58	....	....	......	....
	9/17/19	1562	27,000	100	18,069.69	.669	....	....	......	....

TABLE VIII.—INSIDE FORMS AND INSIDE SCAFFOLDING.

	Date of Figures.	Hull No.	Area in sq. ft of Contact Surface for One Hull.	Per cent of Items Complete.	Total Cost of Items to Date.	Unit Cost of Items per sq. ft. Contact Surface.	Total Man Hours on Item to Date.	Organization of Crews Working on this Item.	Approximate per cent of Each Trade Used on Item.	Average Hourly Wage of Trades.
Jacksonville....	8/27/19	1707	212,181	74	$60,586.36	.39	81,709¾	Carpenters	95	$.80
	8/27/19	1708	212,181	67	54,806.64	.39	73,409	Laborers	5	.46
Mobile........	9/3/19	1715	272,000	100	$85,711.76	.315	114,086⅓	Carpenters	80	$.80
	9/3/19	1716	272,000	100	82,732.29	.30	111,333½	Laborers	20	.46
San Diego..	8/20/19	1723	205,000	79.4	$104,855.87	.644	133,100½	....	......	....
	8/30/19	1724	205,000	63	97,769.60	.755	135.629	....	......	....
San Francisco..	9/24/19	1662	188,000	100	$73,812.37	.389	92,949	....	......	....
	6/22/19	1663	188,000	100	56,417.38	.30	72,025¾	....	......	....
Wilmington....	8/27/19	1560	57,000	100	$54,271.66	.95	....	....	......	....
	9/17/19	1562	57,000	100	32,681.28	.573	....	....	......	....

TABLE IX.—DECK FORMS.

	Date of Figures.	Hull No.	Total Area in sq. ft. Contact Surface for One Hull.	Per cent of Item Complete to Date.	Total Cost of Item to Date.	Unit Cost per sq. ft. Contact Surface.	Total Man Hours on Item to Date.	Organization of Crews Working on this Item	Approximate per cent of Each Trade Used on Item.	Average Hourly Wage of Trades.
Jacksonville....	9/3/19	1707	54,823	60	$10,359.19	.30	14,474	Carpenters	95	$.80
	9/3/19	1708	54,823	38	6,835.23	.25	9,486	Laborers	5	.46
Mobile	9/3/19	1715	54,800	100	$13,905.50	.25	18,676	Carpenters	80	$.80
	9/3/19	1716	54,800	100	14,737.76	.26	17,874½	Laborers	20	.46
San Diego.....	8/20/19	1723	50,000	37	$7,093.75	.381	9,187	....	......	....
	8/20/19	1724	50,000	....	2.40	....	3,000	....	......	....
San Francisco..	8/31/19	1662	55,000	100	$11,823.30	.23	14,245½	....	......	....
	8/31/19	1663	55,000	100	8,985.29	.19	....	....	......	....
Wilmington....	8/27/19	1560	31,680	100	$50,352.03	$1.58	....	....	......	....
	9/17/19	1562	31,680	100	26,354.20	.829	....	....	......	....

TABLE X.—STRIPPING FORMS.

	Date of Figures.	Hull No.	Area in sq. ft. Contact Surface for One Hull.	Per cent of Item Complete.	Total Cost to Date of Item.	Unit Cost of Item per sq. ft. Contact Surface.	Total Man Hours on Item to Date.	Organization of Crews Working on this Item.	Approximate per cent of Each Trade Used on Item.	Average Hourly Wage of Trades.
Jacksonville....	9/3/19	1707	318,728	54	$7,126.47	.04	12,668	Carpenters	50	$.80
	9/3/19	1708	313,728	30	5,055.48	.05	7,770	Laborers	50	.46
Mobile........	9/3/19	1715	380,000	100	$25,573.88	.07	46,105½	Carpenters	10	$.80
	9/3/19	1716	380,000	100	27,848.56	.07	51,041	Laborers	90	.46
San Diego.....	8/20/19	1723	306,000	7.05	$2,731.36	.126	3,979	....	......	....
	8/20/19	1724	306,000	2.45	1,066.82	.142	1,443	....	......	....
San Francisco..	8/31/19	1662	293,000	100	$21,729.38	.074	34,697	....	......	....
	8/31/19	1663	293,000	100	18,380.37	.063	....	....	......	....
Wilmington....	8/27/19	1560	115,680	100	$20,377.68	.17	....	....	......	....
	9/17/19	1562	115,680	96	19,414.77	.17	....	....	......	....

TABLE XI.—CUTTING AND BENDING BARS—STEEL.

	Date of Figures.	Hull No.	Total Quantity in Short Tons.	Per cent of Item Complete to Date.	Total Cost of Item to Date.	Unit Cost of Item per Short Ton.	Total Man Hours on Item to Date.	Organization of Crews Working on Item.	Approximate per cent of Each Trade Used on Item.	Average Hourly Wage of Trades.
Jacksonville....	9/3/19	1707	1,433	85	$5,078.05	4.16	7,784	Blacksmith	50	$.80
	9/3/19	1708	1,433	85	5,078.90	4.16	7,686	Laborers	50	.46
Mobile........	9/3/19	1715	1,483	100	$10,057.10	6.78	16,449	Shearers	50	$.72
	9/3/19	1716	1,483	100	10,057.13	6.78	17,052½	Laborers	50	.46
San Diego.....	8/20/19	1723	1,372	80.6	$6,572.08	5.941	13,015	....	......	....
	8/20/19	1724	1,372	74	5,784.20	5.68	9,437	....	......	....
San Francisco..	8/31/19	1662	753.22	100	$3,624.50	4.81	4,885	....	......	....
	8/31/19	1663	769	100	3,209.63	4.17	....	....	......	....
Wilmington....	8/27/19	1560	440	100	$5,075.70	11.54	....	....	......	....
	9/17/19	1562	440	100	3,980.43	8.06	....	....	......	....

TABLE XII.—CUTTING AND BENDING STIRRUP STEEL.

	Date of Figures.	Hull No.	Total Quantity for One Hull in Short Tons.	Per cent of Item Complete.	Total Cost of Item to Date.	Unit Cost of Item per Short Ton.	Total Man Hours on Item to Date.	Organization of Crews Working on Items.	Approximate per cent of Each Trade Used on Item.	Average Hourly Wage of Trades.
Jacksonville..	9/3/19	1707	137	85	$3,124.78	26.80	4,710	Steel workers	50	$.80
	9/3/19	1708	137	85	3,119.55	27.37	4,740	Laborers	50	.46
Mobile......	9/3/19	1715	137	100	$5,611.15	40.95	9,662½	Laborers	50	$.46
	9/3/19	1716	137	100	5,611.09	40.95	9,664	Shearers	50	.72
San Diego...	8/20/19	1723	100	99.3	$2.940.77	29.606	4,790	....	......	....
	8/20/19	1724	100	96.1	2,636.58	27.42	4,192	....	......	....
San Francisco	8/31/19	1662	77.22	100	$1,579.71	20.45	1,811	....	......	....
	8/31/19	1663	85	100	1,566.37	18.43	....	....	......	....
Wilmington..	8/27/19	1560	52	100	$2,726.76	42.85	....	....	......	....
	9/17/19	1562	52	100	2,641.53	35.69	....	....	......	....

TABLE XIII.—FABRICATING AND PLACING SHELL REINFORCEMENT.

	Date of Figures.	Hull No.	Total Quantity for One Hull in Short Tons.	Per cent of Item Complete.	Total Cost of Item to Date.	Unit Cost of Item per Short Ton.	Total Man Hours on Item to Date.	Organization of Crews Working on Item.	Approximate per cent of Each Trade Working on Item.	Average Hourly Wage of Trades.
Jacksonville..	9/3/19	1707	444	90	$9,672.45	24.31	19.910	Steel workers	10	$.80
	9/3/19	1708	444	96	11,122.62	25.92	19,560	Machinists	5	.80
								Laborers	85	.46
Mobile......	9/3/19	1715	428	100	$12,227.28	28.37	22,827	Steel workers	50	.60
	9/3/19	1716	428	100	14,128.84	33.01	26,675½	Laborers....	50	.46
San Diego...	8/20/19	1723	435	96.5	$13,436.96	32.14	18,545	....	......	....
	8/20/19	1724	430	95	10,382.86	25.35	14,635	....	......	....
Oakland.....	8/31/19	1662	478.9	100	$7,251.09	15.15	8,139	....	......	....
	8/31/19	1663	483	100	6,497.66	13.43	....	....	......	....
Wilmington..	8/27/19	1560	120	100	$12,260.10	102.17	....	....	......	....
	9/17/19	1562	120	100	7,538.27	62.82	....	....	......	....

TABLE XIV.—FABRICATING AND PLACING INSIDE REINFORCEMENT.

	Date of Figures.	Hull No.	Total Quantity for One Hull in Short Tons.	Per cent of Item Complete to Date.	Total Cost of Item to Date.	Unit Cost of Item per Short Ton.	Total Man Hours on Item to Date.	Organization of Crews Working on Item.	Approximate per cent of Each Trade on Item.	Average Hourly Wage of Trades.
Jacksonville....	9/3/19	1707	609	99.8	$41,677.53	66.61	73,730	Steel workers	10	$.80
	9/3/19	1708	609	94	35,403.17	61.73	64,760	Machinists	5	.80
								Laborers	85	.46
Mobile........	9/3/19	1715	624	100	$30,157.31	48.33	53,620½	Reinforcing	50	$.60
	9/3/19	1716	624	100	25,345.69	40.62	43,701½	Steel workers		
								Laborers	50	.46
San Diego.....	8/20/19	1723	640	92.7	$31,943.10	53.82	42,334	....	......	....
	8/20/19	1724	640	88	26,860.23	47.70	34,906	....	......	....
San Francisco..	8/31/19	1662	515	100	$15,630.89	30.32	17,846½	....	......	....
	8/31/19	1663	540	100	14,699.47	27.20	....	....	......	....
Wilmington....	8/27/19	1560	260	100	$12,962.36	49.46	....	....	......	....
	9/17/19	1562	260	100	13,191.55	50.67	....	....	......	....

TABLE XV.—FABRICATING AND PLACING DECK REINFORCEMENT.

	Date of Figures.	Hull No.	Total Quantity for One Hull in Short Tons.	Per cent of Item Complete to Date.	Total Cost of Item to Date.	Unit Cost of Item per Short Ton.	Total Man Hours on Item to Date.	Organization of Crews Working on Item.	Approximate per cent of Each Trade on Item.	Average Hourly Wage of Trades.
Jacksonville....	9/3/19	1707	553	12	$3,586.00	54.08	11,625	Steel workers	10	$.80
	9/3/19	1708	553	17	5,628.65	60.59	6,370	Machinists	5	.80
								Laborers	85	.46
Mobile........	9/3/19	1715	530	100	$13,664.38	25.78	18,216	Reinforcing	70	$.60
	9/3/19	1716	530	100	15,045.04	23.38	22,196½	Steel workers		
								Laborers	30	.46
San Diego.....	8/20/19	1723	402	9.7	$2,052.61	52.59	2,242	....	..	..
	8/20/19	1724	402	.....	....	......	......	....	..	
San Francisco..	8/31/19	1662	434	100	$8,215.42	18.91	9,202	....	......	....
	8/31/19	1663	419	100	8,334.18	19.90	......	....	......	....
Wilmington....	8/27/19	1560	134	100	$9,426.58	69.37	......	....	......	....
	9/17/19	1562	134	100	8,555.43	63.84	......	....	......	....

TABLE XVI.—SETTING SLEEVES, INSERTS, ETC.

	Date of Figures.	Hull No.	Total Quantity for One Hull in lb.	Per cent of Item Complete to Date.	Total Cost of Item to Date.	Unit Cost of Item per lb.	Man Hours on Item to Date.	Organization of Crews Working on Items.	Approximate per cent of Each Trade.	Average Hourly Wage of Trades.
Jacksonville....	9/3/19	1707	208,000	21	$7,066.93	.16	9,013	Carpenters	20	$.80
	9/3/19	1708	208,000	20	6,453.24	.15	8,365	Machinists	60	.80
								Laborers	20	.46
Mobile........	9/3/19	1715	42,000	100	$8,669.24	.20	12,050½	Pipe fitters	80	$.80
	9/3/19	1716	42,000	100	9,059.11	.21	12,423½	Labor	20	.46
San Diego.....	8/20/19	1723	91,500	57.2	$11,756.58	.2459	13,580	....	....	....
	8/20/19	1724	91,500	38	9,002.32	.259	11,347½	....	....	....
San Francisco..	8/31/19	1662	50,000	100	$11,447.24	.229	13,264¼	....	....	...
	8/31/19	1663	50,000	100	12,606.34	.252	....	....	....	....
Wilmington....	8/27/19	1560	5,000	100	$7,455.26	1.49	......	....	....	....
	9/17/19	1562	7,000	98	6,485.36	.95	....	....	....	....

TABLE XVII.—SETTING STERN FRAME AND PLATING.

	Date of Figures.	Hull No.	Quantity for One Hull in lb.	Per cent of Item Complete.	Total Labor Cost of Item to Date.	Unit Cost of Item per lb.	Total Man Hours on Item to Date.	Organization of Crews Working on Items.	Approximate per cent of Each Trade of Crews.	Average Hourly Wage of Trades.
Jacksonville....	9/3/19	1707	28,000	100	$882.70	.03	1,455	Riggers	5	$.74
	9/3/19	1708	28,000	100	1,032.53	.04	1,247	Machinists	90	.80
								Laborers	5	.46
Mobile . .	9/3/19	1715	36,000	100	$1,790.19	.05	2,571½	Riveters		
	9/3/19	1716	36,000	100	1,933.34	.05	2,934½	Machinists	70	$.80
								Laborers	30	.46
San Diego . .	8/20/19	1723	49,670	100	$4,348.37	.09	5,193	....	....	..
	8/20/19	1724	49,670	100	3,476.55	.07	3,860	....	....	..
San Francisco	8/31/19	1662	30,000	100	$1,184.62	.03	1,166			..
	8/31/19	1663	39,000	100	927.27	.024	.....	. .	.	..
Wilmington ...	8/27/19	1560	39,000	100	$3,045.81	.078	.....	. . .	.	...
	9/17/19	1562	39,000	100	2,917.49	.07	.....	...		

TABLE XVIII.—MIXING AND PLACING CONCRETE.

	Date of Figures.	Hull No.	Total Quantity per Hull in cu. yd.	Per cent of Item Complete.	Total Labor Cost of Item to Date.	Unit Cost of Item per cu. yd.	Total Man Hours on Item to Date.	Organization of Crews Working on Item.	Approximate per cent of Each Trade on Crews.	Average Hourly Wage of Trades.
Jacksonville....	9/3/19 9/3/19	1707 1708	2,793 2,793	67 57	$64,678.81 48,013.85	34.56 30.38	116,755 88,356	Laborers Carpenters Riggers	90 5 5	$.46 .80 .74
Mobile........	9/3/19 9/3/19	1715 1716	2,880 2,880	100 100	$69,167.71 71,074.70	23.32 24.67	115,848 124,979	Laborers Finishers	80 20	$.46 .72
San Diego.....	8/20/19 8/20/19	1723 1724	2,790 2,790	73.7 38	$28,735.26 16,090.51	13.94 15.02	49,917 22,807	....	....	....
San Francisco..	8/31/19 8/31/19	1662 1663	2,726.5 2,900	100 100	$56,949.55 58,575.95	20.88 18.13	72,353½	....	....	....
Wilmington....	8/27/19 9/17/19	1560 1562	1,208 1,208	100 100	$42,993.89 30,104.20	35.59 24.92	....	....	....	....

TABLE XIX.—PAINTING AND COATING HULL.

	Date of Figures.	Hull No.	Total Area to be Covered, One Hull.	Per cent of Item Complete.	Total Cost of Item to Date.	Unit Cost of Item per sq. ft.	Total Man Hours on Item to Date.	Organization of Crew Working on Item.	Approximate per cent of Each Trade on Item.	Average Hourly Wage of Trades.
Jacksonville....	9/3/19 9/3/19	1707 1708	....	....	....	....	1,926 1,476	Painters Laborers	95 5	$.80 .46
Mobile........	9/3/19 9/3/19	1715 1716	159,200 270,000	100 100	$1,268.61 2,610.15	.08 .097	2,176 5,026	Painters	100	$.80 ....
San Diego.....	8/20/19 8/20/19	1723 1724	....	....	....	....	....	....	....	....
San Francisco..	8/31/19 8/31/19	1662 1663	50,000 50,000	100 100	811.01 830.18	.016 .016	1,223½	....	....	....
Wilmington...	8/27/19 9/17/19	1560 1562	115,680 115,680	100 19	$6,360.38 1,612.00	.05 .07	....	....	....	....

TABLE XX.—LAUNCHING.

	Date of Figures.	Hull No.	Total Quantity of Packing in ft. B. M.	Per cent of Item Complete to Date.	Total Labor Cost of Item to Date.	Unit Cost of Item per ft. B. M.	Total Man Hours on Item to Date.	Organization of Crews Working on Items.	Approximate per cent of Each Trade on Item.	Average Hourly Wage of Trades.
Jacksonville....	9/3/19	1707	297,597	10	$1,223.57	......	1,545	Carpenters	80	$.80
	9/3/19	1708	....	......	....	......	3,800	Laborers	20	.46
Mobile.........	9/3/19	1715	250,000	100	$8,027.57	.03	8,793	....	......	....
	9/3/19	1716	250,000	100	17,219.14	.06	22,541	....	......	....
San Diego.....	8/20/19	1723	....	......	....	......	....	....	......	....
	8/20/19	1724	....	......	....	......	....	....	......	....
San Francisco..	8/31/19	1662	....	75	$11,939.91	......	14,641¾	....	......	....
	8/31/19	1663	....	100	17,658.36	......	....	....	......	....
Wilmington....	8/27/19	1560	190,000	100	$23,136.49	.12	....	....	......	....
	9/17/19	1562	190,000	52	6,181.28	.06	....	....	......	....

to give more rapid progress was stopped, overtime work was discontinued, and lower costs made the paramount issue in construction. This naturally slowed up the rate of construction, and the curves shown are to be interpreted in this light. Another item which prevented fast construction was the inexperience of everyone concerned in the work. At Mobile and San Francisco, a third ship is now being built by the same organization which have already completed two others. These are hull numbers 1,664 and 1,717. Both are now about half complete, and if the average rate of construction is maintained, they will be built in about 60 per cent of the time of the first ships at these yards.

STATUS OF CONCRETE SHIPS.

The concrete ship is a practical structure and if properly designed and built will resist the normal stresses to which ships are exposed at sea.

It has, however, one inherent weakness. The 4-in. shell of a concrete ship will not resist local impact of moderate intensity to the same extent as the shell of a steel ship of the same capacity. Tests were made on both concrete and steel panels designed to duplicate a section of the shell of a concrete and steel ship of similar size. These tests showed that impact loads applied between frames would only dent the shell of a steel ship and possibly loosen some rivets while loads of similar intensity would shatter the concrete shell between frames. In cases of severe impact where their resistance of the frames is an important factor it is believed that the concrete ship will show equal if not greater resistance than a steel ship on account of its greater mass. Up to the present writing, no concrete ships have been exposed to collision or severe impact; therefore, the behavior of a concrete ship under such conditions is unknown. As a compensating feature, it has been found

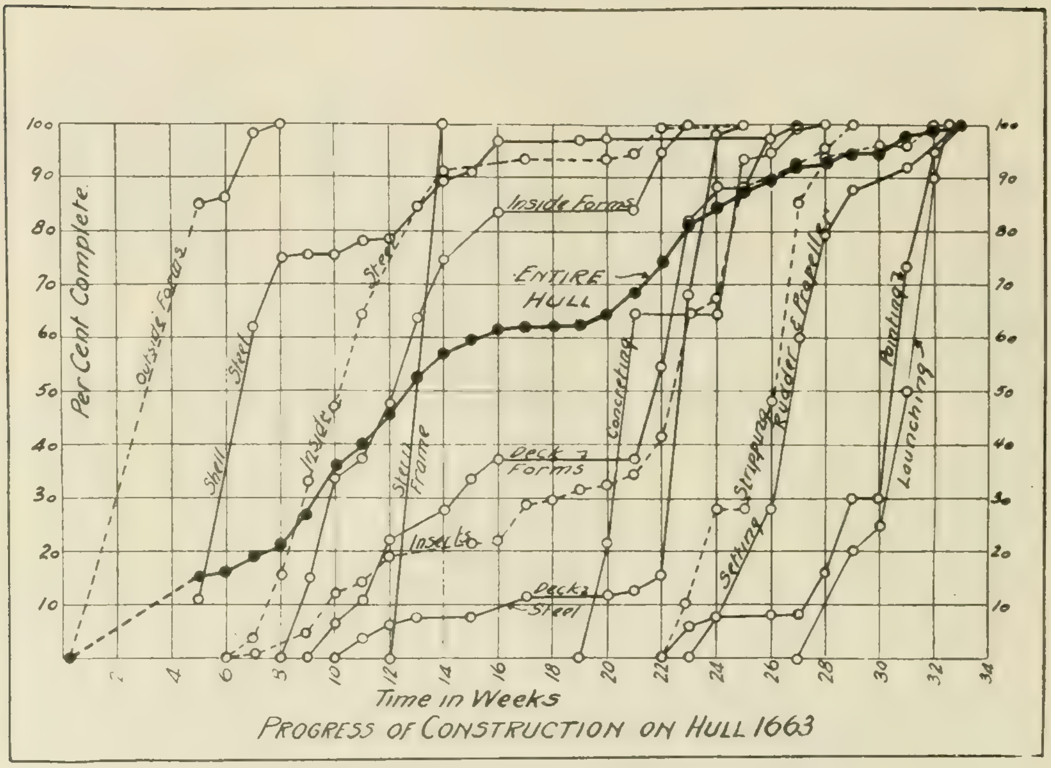


FIG. 44.—TIME AND PROGRESS CURVES OF CONSTRUCTION ON HULL 1663.

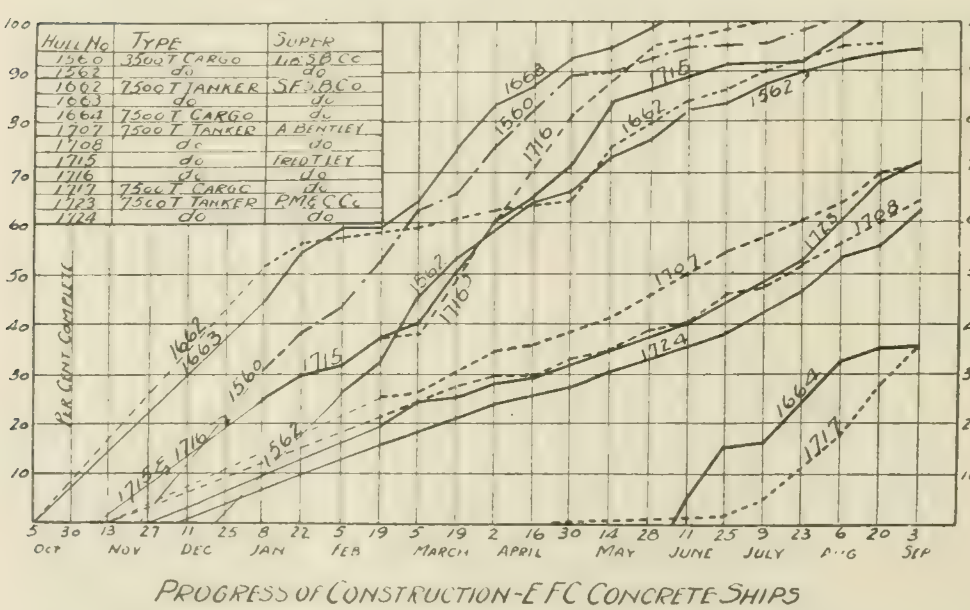


FIG. 45.—TIME AND PROGRESS CURVES ON ALL EMERGENCY FLEET CORPORATION CONCRETE SHIPS.

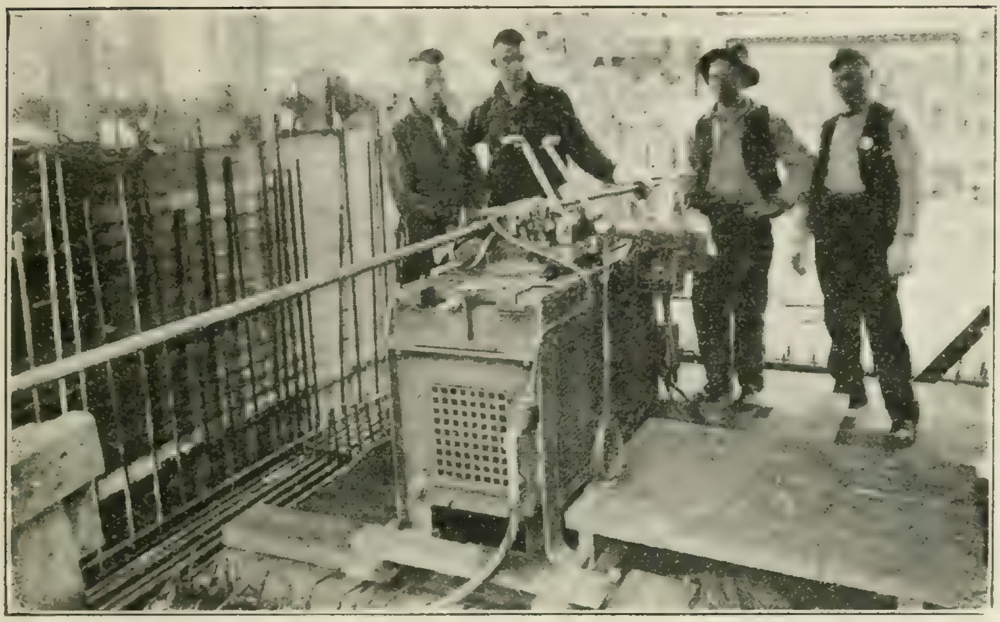


FIG. 46.—RESISTANCE BUTT WELDER USED AT WILMINGTON.

UNITED STATES SHIPPING BOARD
EMERGENCY FLEET CORPORATION
DIVISION STEEL SHIP CONSTRUCTION
CONCRETE SHIP SECTION

WEEKLY LABOR COST AND PROGRESS REPORT

SUPERINTENDENT

WEEK ENDING
HULL NO.

LABOR COST						PROGRESS OF WORK						
ITEM	PAY ROLL				UNIT LABOR COST	QUANTITY				PER CENT ERECTED %	UNIT VALUE	VALUE COMPLETE
	HOURS	THIS WEEK		TOTAL TO DATE		THIS WEEK		ERECTED TO DATE	TOTAL QUANTITY FOR HULL			
		MAKING & REPAIRING	ERECTING			MADE	ERECTED					
Outside Scaffolding, Blocking, Roof Trusses												
Outside Forms												
Inside Forms and Inside Scaffolding												
Deck Forms												
Stripping Forms and Removing Scaffolding												
Cutting and Bending Bar Steel												
Cutting and Bending Stirrup Steel												
Fabricating and Placing Shell Reinforcement												
Fabricating and Placing Inside Reinforcement												
Fabricating and Placing Deck Reinforcement												
Setting Sleeves, Inserts, Sockets, Etc.												
Setting Stern Frame and Plating												
Mixing and Placing Concrete												
Painting and Coating Hull												
Launching												
Setting Propellers, Shaft and Rudder												
Distributed Prehandling Expense —Lumber												
" " —Steel												
" —Cement												
" " —Aggregate												
Templets and Molds												
TOTALS												

NOTE.—For the Direct Material Cost, Salvage Debits and Credits, Proportion of General Administrative, Engineering and Yard Operation Expenses, etc., see monthly summary report.

NUMBER OF MEN EMPLOYED
FOREMEN STEEL WORKERS
CARPENTERS LABORERS

WEEK ENDING
HULL NO.

FIG. 47.—FORM USED FOR REPORTING WEEKLY LABOR COST AND PROGRESS.

that concrete ship hulls are more easily and cheaply repaired than steel ship hulls.

On account of this more friable character of the shell of a concrete ship, it is necessary that it be handled more carefully than a steel ship when in harbor. It is possible some means will be found to obviate this weakness.

The future utility of the concrete ship depends upon two factors. One, its durability and the other its economy of operation as compared with wood and steel ships.

The final durability of the concrete ship can only be determined by years of experience but it is quite probable that two or three years of operation of the ships now building will furnish some indication of the life which may be anticipated.

The economy of the concrete ship as compared with steel and wood ships assuming that it will have an equal life depends upon the relative first cost and the difference in the cost of propulsion due to the variation in the tare weight of the hulls for equal carrying capacity. The ratio of the dead weight carrying capacity to the total displacement for the lightest concrete ship so far constructed is .56, which is about equal to the wood ship and much less than the steel ship. For ships of the same dimensions the concrete ship has a greater capacity for measurement cargo than the wood ship and only a slightly less capacity than the steel ship. For ships of the same dead weight the concrete ship has much greater capacity for measurement cargo than either steel or wood ships. The relative economy of the several types of ships will, of course, vary depending upon the type of cargo handled, length of voyage and percentage of time in port.

When the concrete ship has demonstrated its durability, many situations will be found where it will prove more economical than either wood or steel in spite of its greater hull weight which probably can never be entirely overcome.

CONSTRUCTION OF CONCRETE BARGES FOR USE ON NEW YORK STATE BARGE CANAL.

S. C. HOLLISTER.*

In the spring of 1918, at the request of the Committee on Inland Waterways, of the United States Railroad Administration, the Concrete Ship Section of the Emergency Fleet Corporation undertook the design of a standard reinforced-concrete canal barge for use by the Railroad Administration on the New York State Barge Canal. The barge was to have a carrying capacity of 500 long tons; was to meet the requirements of form and dimension of the New York State Barge Canal Regulations, and was to have such exterior form as to be easily propelled at six miles an hour. The barge was to be without power equipment, but was to be provided with means of towing and steering.

To meet these requirements the Section prepared a design of a barge 150 ft. long, having a beam of 21 ft. and a molded depth amidships of 12 ft. The barge was to carry 500 long tons on a load draft of 10 ft. Because of the rather small freeboard the barge was given a sheer of $1\frac{1}{2}$ ft. aft and $2\frac{1}{2}$ ft. forward. The lines of the barge were chosen such that the ends were shipshape with a parallel middle-body of 120 ft. The rudder was designed to be supported outside of the overhang of the stern.

The interior of the hull is divided into three main compartments. The forward 15 ft. provides crew's quarters and storage space. It is separated from the main cargo hold by a transverse water-tight concrete bulkhead, which also serves as a collision bulkhead. The main cargo hold is 115 ft. long, and is provided with a hatchway 14 ft. wide in the clear by 105 ft. long, interrupted every 15 ft. by an 8 x 12-in. transverse hatch beam. The after end of this cargo hold is formed by a timber bulkhead which separates it from the barge captain's quarters.

Structurally, the canal barge is of the frame type, the frames being spaced 5 ft. apart on centers. The bottom members of these frames are 18 in. deep from the exterior of the shell to the top of the rib, and the side members are 12 in. deep from the exterior of the hull to the inside face of the rib. Every third frame is made continuous around the girth of the barge to provide the transverse hatch beam just referred to. These frames have a width of 8 in. The intermediate frames are 6 in. wide and extend from one side of the hatchway around the girth of the boat to the other side. Center and side keelsons, at the center and quarter points, respectively, of the span of the bottom member of the transverse frames, extend the full length of the bottom of the barge.

Amidships the thickness of the shell of the barge is 3 in. The forward curved surface of the shell is $4\frac{1}{2}$ in. thick and the after curved surface 4 in. thick. The deck in general has a thickness of 3 in. The shell is reinforced

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transversely on the bottom and vertically on the sides, by ½-in. square bars spaced 7 in. on centers. The longitudinal steel in the bottom consists of ½-in. square bars spaced 3 in. on centers on each face of the slab. This is changed on the side walls to ⅜-in. square bars at variable spacing. The deck slab is reinforced longitudinally with ½-in. bars spaced 4 in. on centers on either face. At the junction of the side wall with the deck a triangular longitudinal beam is reinforced with seventeen ⅞-in. square bars. The barge is designed for a longitudinal bending moment of $WL/40$ and a total transverse shear of

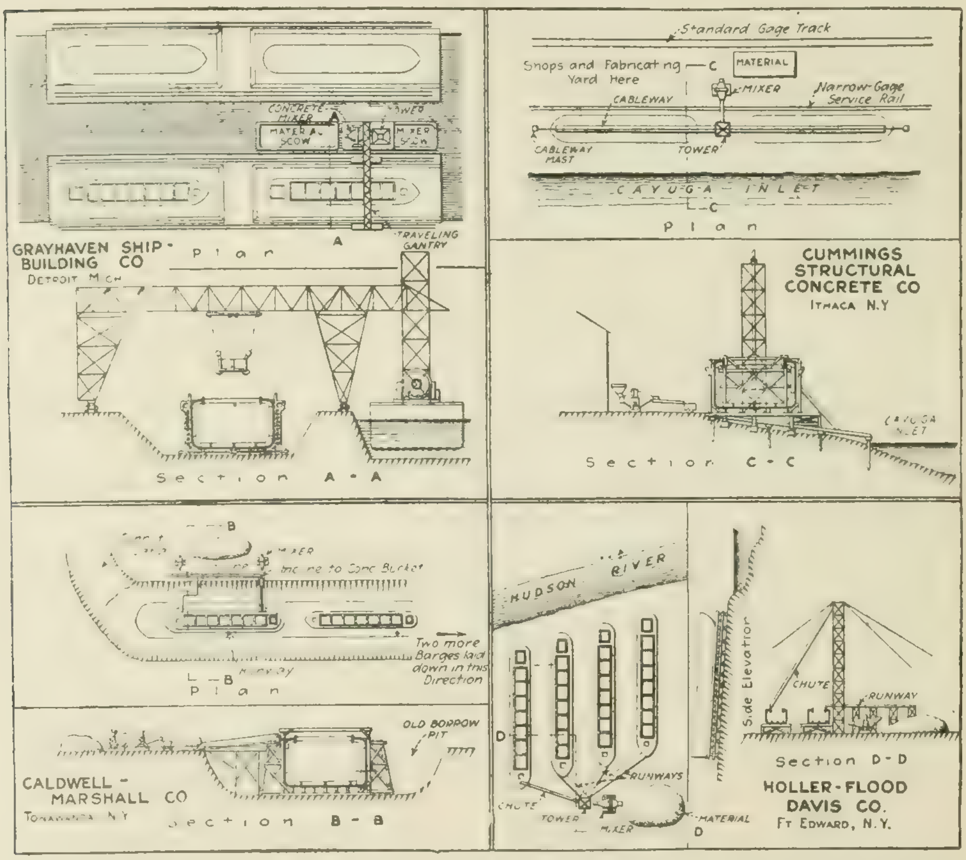


FIG. 1.—LAYOUT OF FOUR YARDS WHERE NEW YORK STATE CANAL 500-TON CONCRETE BARGE IS BEING BUILT.

$W/13$ in which formulas W represents the total load displacement of the barge, and L the over-all length of the hull.

The interiors of the forward and aft cabins are ceiled with ⅞-in. matched ceiling. The after cabin is divided by partitions into a living room, bedroom, galley and wardrobe. The galley cabinet is built-in. Water is supplied at a sink from a 100-gallon steel tank arranged for filling from the deck. The cargo hold is lined with matched ceiling 3 in. thick on the bottom and 2 in. thick on the side. Sections of the bottom and sides of this lining are removable to permit access to the space between it and the shell of the barge. The main hatchway is provided with transverse sectional hatch covers which

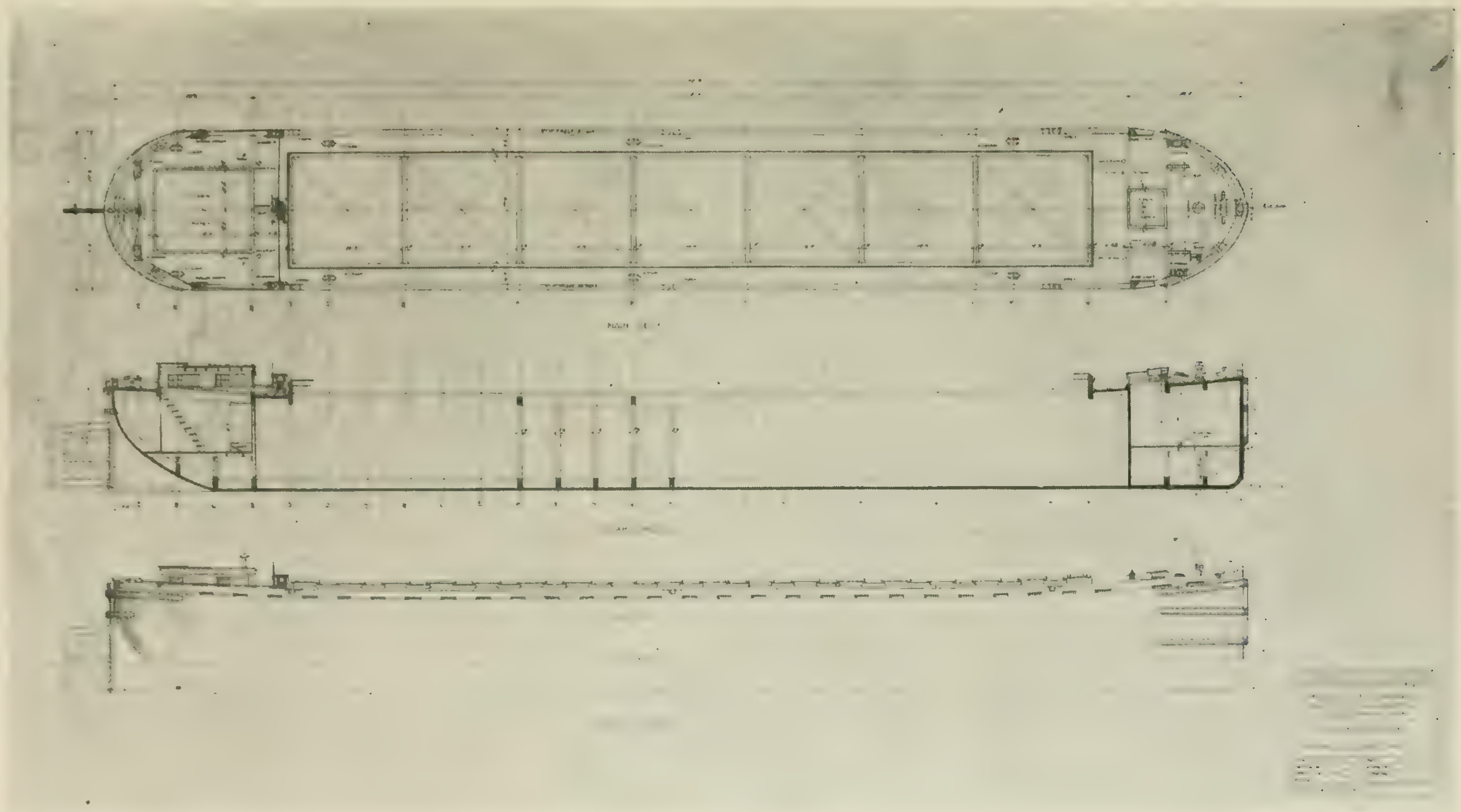


FIG. 2.—GENERAL DRAWING OF 500-TON CONCRETE BARGE FOR NEW YORK STATE BARGE CANAL.

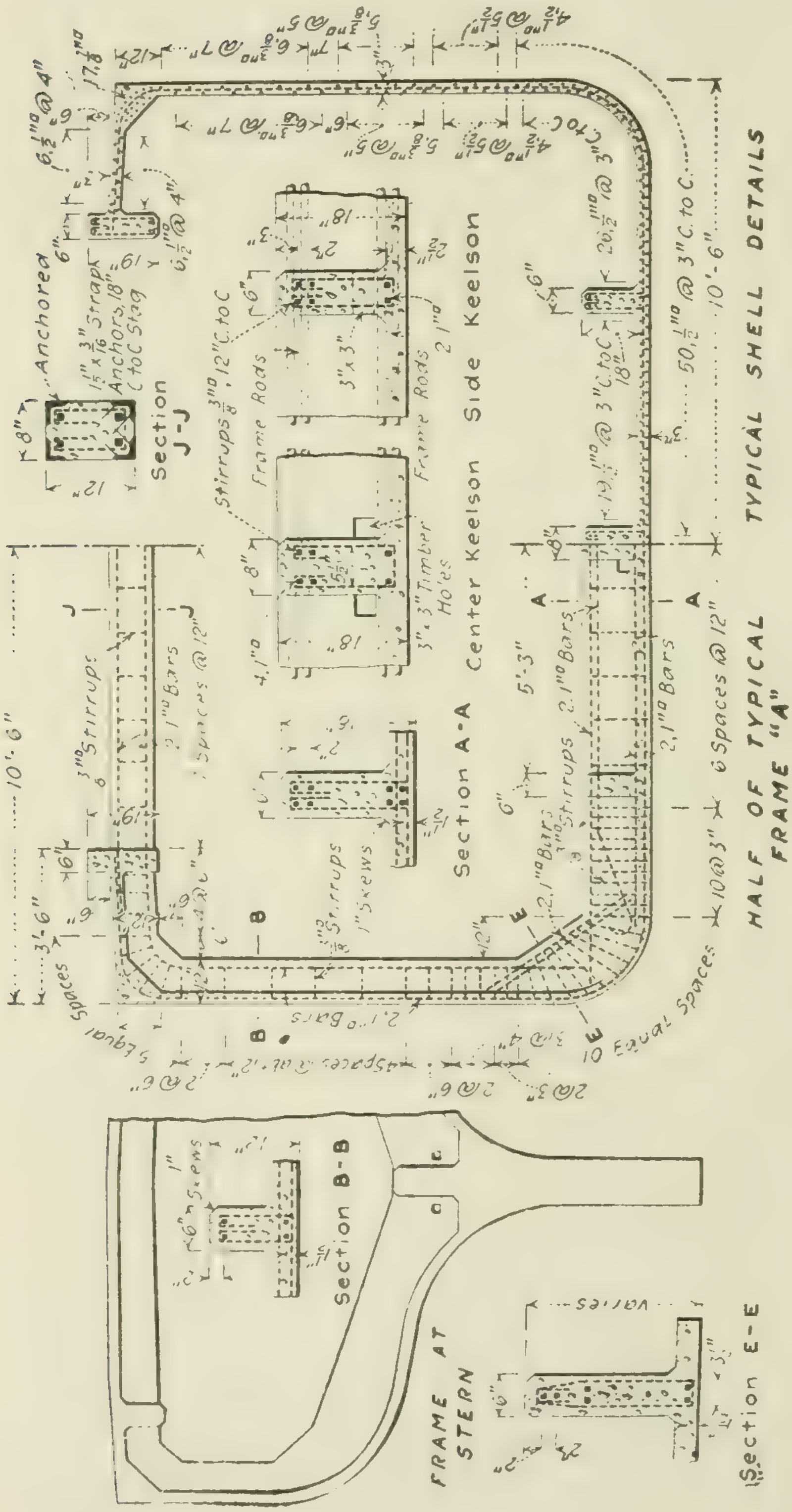


FIG. 3.—SECTION OF REINFORCED CONCRETE GIRDER FRAMES AND THREE-INCH SHELL

can be handled conveniently by two men. A sectional canvas cover is provided to extend over the entire top of the main hatch and is battened and laced in place.

The cargo hold is equipped with four 10-in. square box siphon wells, which extend from the deck to the space beneath the hold floor, for use in emergencies when a steam suction is necessary. It is also fitted with four 3-in. hand operated diaphragm pumps. A similar diaphragm pump is installed forward of the collision bulkhead in the crew's quarters.

The barge is equipped with necessary chocks and cleats for convenient handling. A large hollow casting filled with concrete, into which reinforcing has been extended from the hull framing, forms the towing bitt. A 30-in. crank capstan is provided for assistance in handling. The equipment includes a 500-lb. anchor with davit for handling it. The barge is steered by a double-drum Gillie steering wheel mounted just forward of the aft cabin. It operates

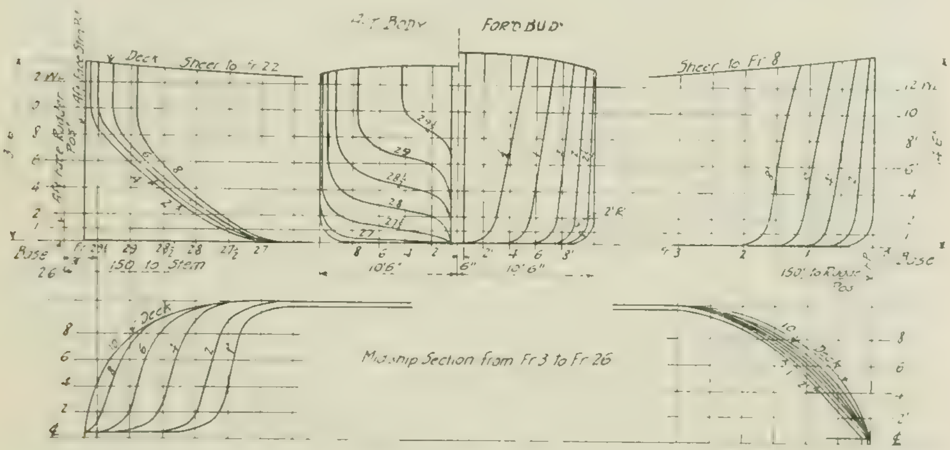


FIG. 4.—LINES OF 500-TON CONCRETE BARGE.

a $\frac{3}{4}$-in. plow-steel cable which may be connected either to the tiller, or, by means of suitable sheaves, to a barge being towed immediately aft. This latter arrangement makes it possible to bend a line of barges in tow to facilitate steering around sharp curves in the canal.

The construction of twenty-one barges upon these plans has been carried on in four yards, the first of which is at Fort Edward, New York, building eight barges; the second at Ithaca, building four barges; the third at Tonawanda, also building four barges; and the fourth building the remaining five barges at Detroit, Michigan. Each builder used his own system of construction, plant layout and form details. The widely varied methods in each of the yards has given excellent opportunity for a wide range of experience in this form of construction.

The Fort Edward yard, operated by the Holler-Davis-Flood Co., consists of four ways constructed after the usual manner of an end-launching shipyard. Because the Hudson River at this point is quite narrow, these ways are inclined about 45 degrees to the center line of the stream. The declivity of the ways is $\frac{3}{4}$ in. per ft.

The outside forms for the shell are made in sections 30 ft. long, each section providing the surface forming for the shell from the center line of the keel to the top of the side wall. These sections are carried on blocking until after concreting is accomplished, after which upon removal of the blocking they are dropped on rollers and drawn away from the side of the barge. At the ends a quadrant of the curved surface was erected on a single section.

The inside forms were built in sections, and when assembled were supported from trusses which rested on the vertical supports of the outside forms. The forms for the under side of the deck likewise were supported from these same trusses. Reinforcing steel was bent upon a large platform constructed for the purpose.

The plant layout includes the woodworking shop, the bending platform,

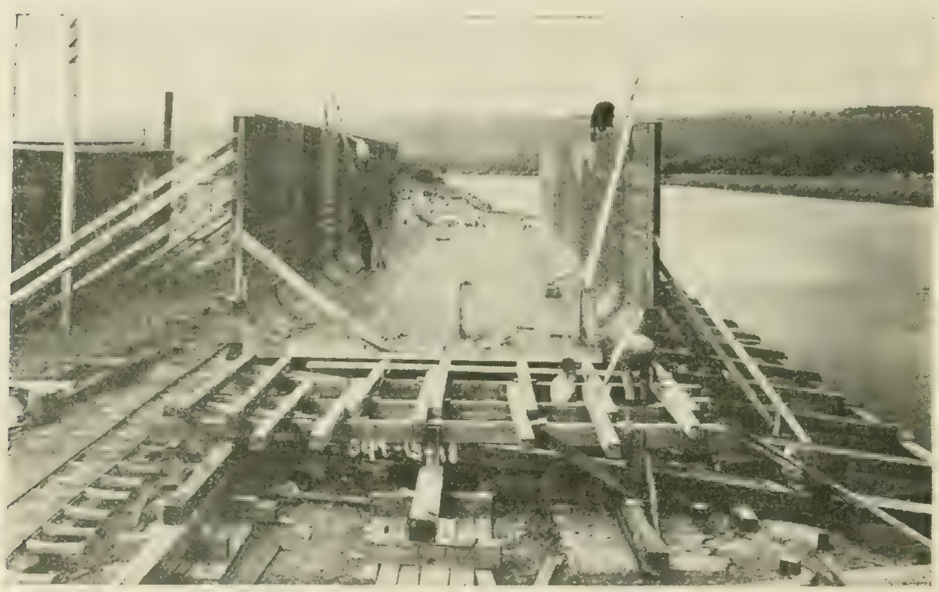
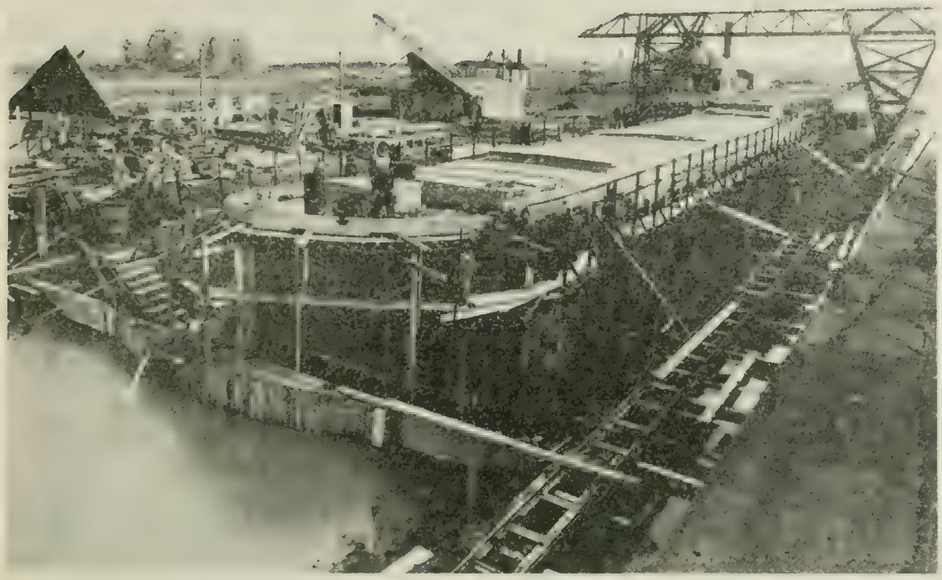


FIG. 5.—OUTSIDE FORM ON SIDE LAUNCHING WAYS, ITHACA YARD.

the supplies shed, cement storage shed, blacksmith shop, office, reinforcement storage racks, and the concreting plant. The one-yard steam-driven Ransome mixer, handling about one-half a batch at a time, was ample to supply the concrete as rapidly as was desired. The plant also had an air compressor for supplying the air hammers used in vibrating the forms.

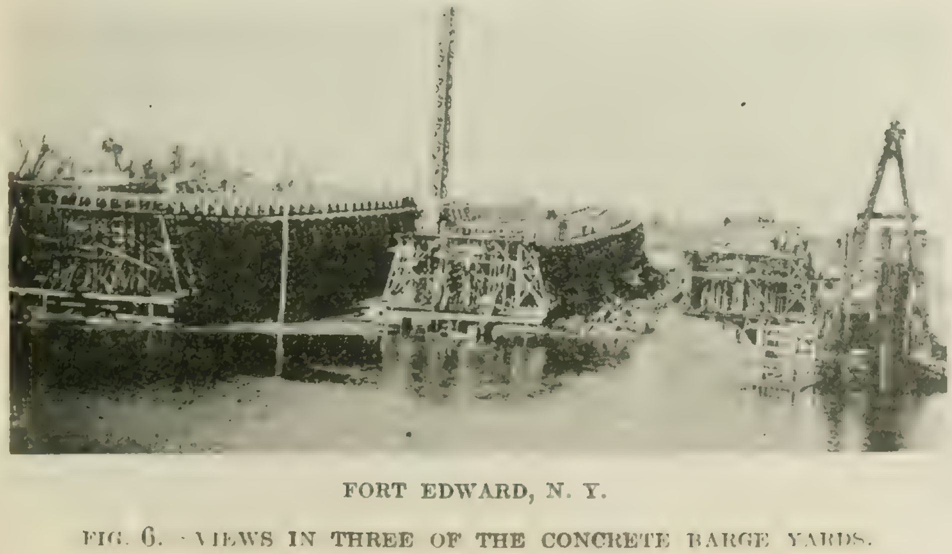
The yard at Ithaca, New York, operated by the Cummings Structural Concrete Co. of Philadelphia, was located on the inlet to Lake Cayuga, and consisted of two ways arranged for sidewise launching. A small cable-way was placed over these ways for handling the necessary materials during construction. A barge during construction was supported on bents set immediately between the launching ways.

The outside forms in this yard were constructed in sections extending 10 ft. along the length of the barge in each case. The bottom forms extended across the flat portion of the bottom. These were joined by the side forms to



DETROIT, MICH.

ITHACA, N. Y.



FORT EDWARD, N. Y.

FIG. 6.—VIEWS IN THREE OF THE CONCRETE BARGE YARDS.

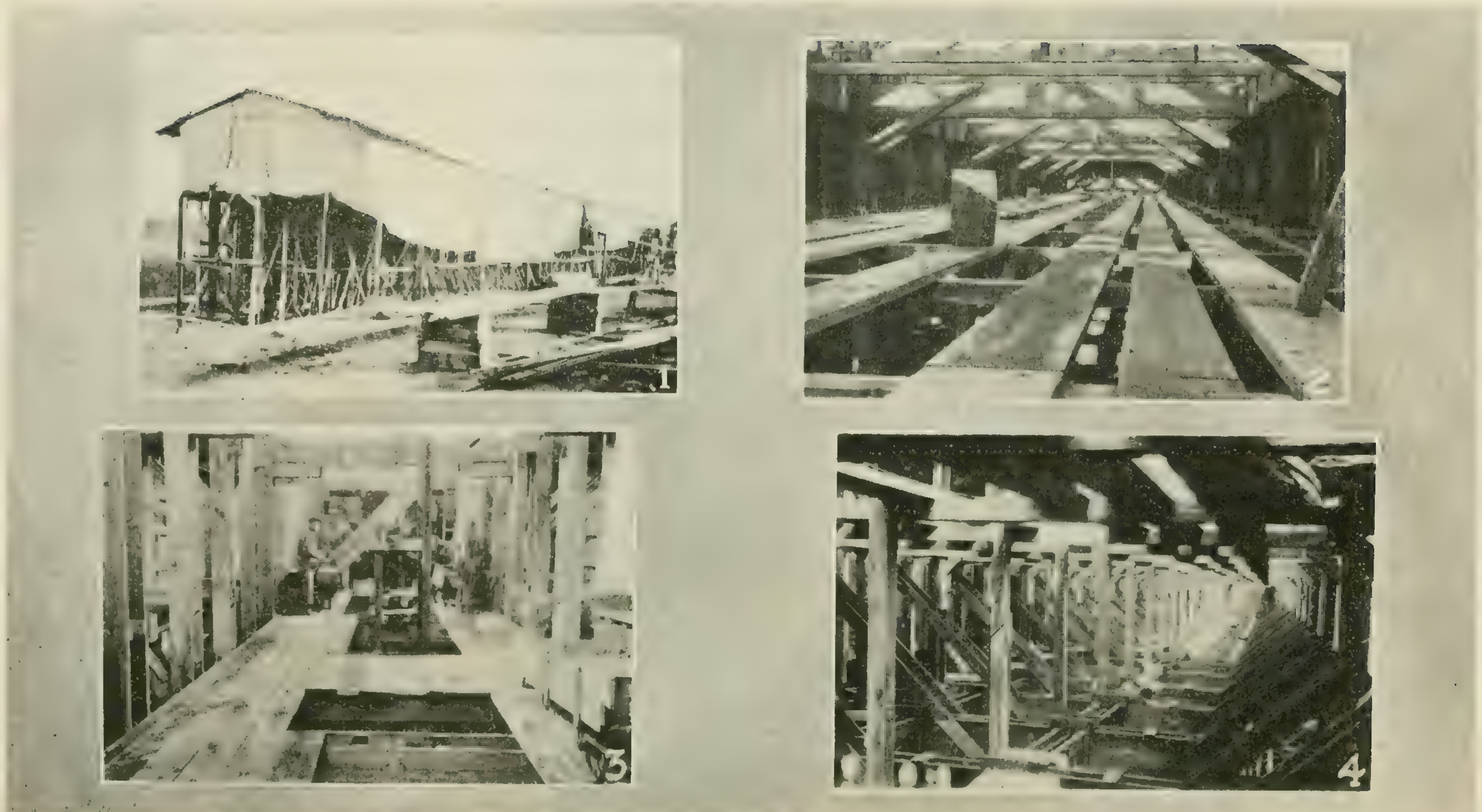


FIG. 7.—1 AND 3, PROTECTION COVER AND WORKING PLATFORM, TONAWANDA.
2 AND 4, WORKING PLATFORM FOR CONCRETING, DETROIT.

which the bilge curve was attached. The quadrants of the curved portions of the barge were made in single sections. The sheathing on these curved portions in this yard was of cypress.

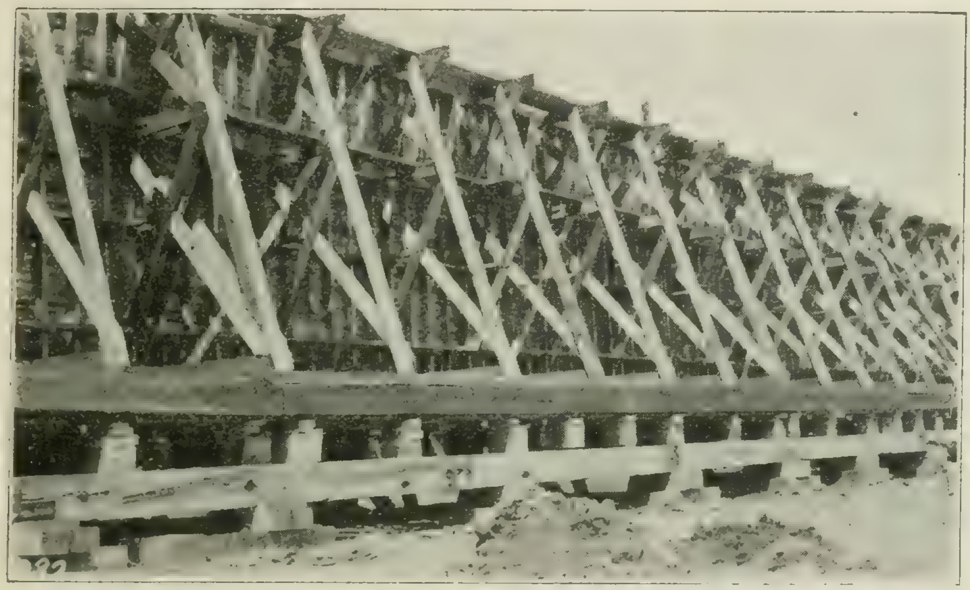


FIG. 8.—OUTSIDE FORMS IN FORT EDWARD YARD.

FIG. 9.—BARGE FORMS IN TONAWANDA YARD.

The inside forms were made in units carried on transverse trusses which rested on 6 by 6 by 3-in. concrete blocks, which later became incorporated in the bottom slab of the barge. The forms for the inside face of the shell

extended from one transverse rib to the next, and were provided with two vertical rows of windows spaced 18 in. apart in a vertical direction for the purpose of introducing the concrete.

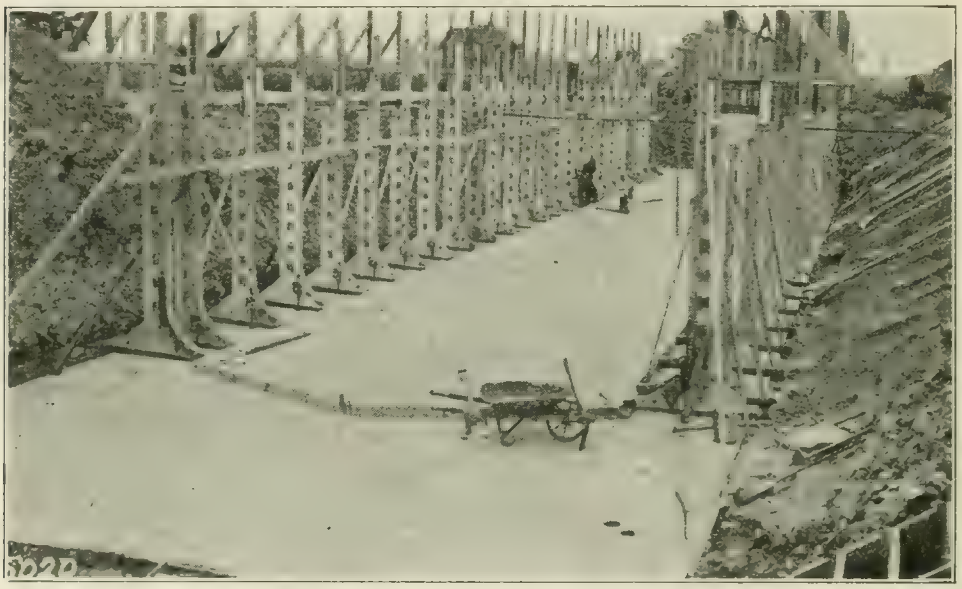


FIG. 10.—CONCRETE SIDE FRAMES FOR FORMS AT DETROIT YARD.

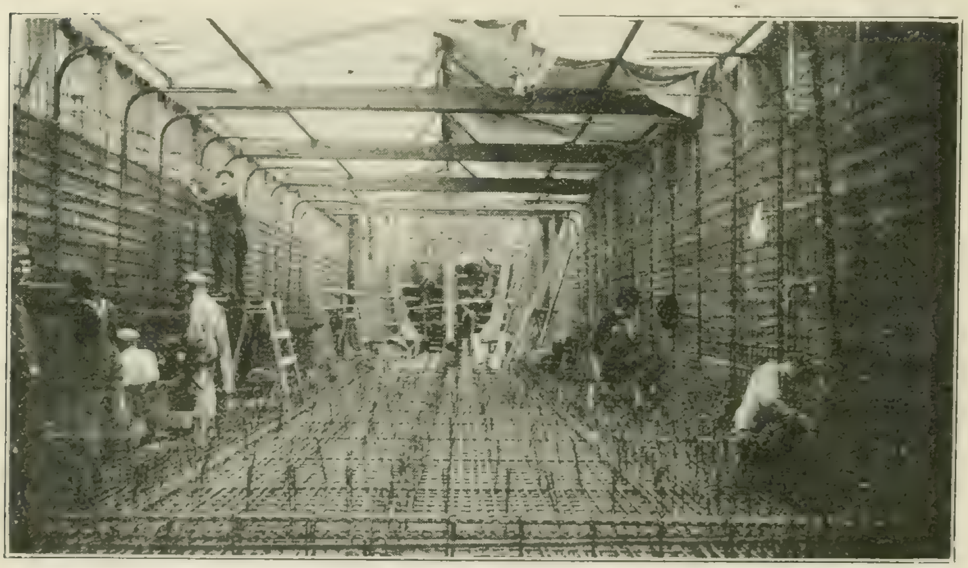


FIG. 11.—PLACING REINFORCEMENT IN SHELL, TONAWANDA YARD.

The yard layout included the woodworking shop, cement storage shed, blacksmith ship and office. The concrete plant consisted of a $\frac{3}{4}$-yd. Ransome mixer with double platform hoist. Two Westinghouse air-brake compressors furnished the necessary air for the vibrating hammers.

The yard at Tonawanda, operated by the Caldwell-Marshall Co. of Columbus, Indiana, was located along the right-of-way of the electric high-speed line between Buffalo and Niagara Falls. The barges were constructed in a long basin located in the borrow pit along this right of way. Launching was accomplished by floating these barges in this basin and towing them into Ellicott Creek, which is a feeder of the New York Barge Canal.

The barges during construction were supported on low transverse bents in such a manner that the bottom of the barge was from 18 in. to 2 ft. above the bottom of the basin.

The outside forms were constructed continuously upon vertical jacks which were braced to the transverse joists of the under supports. The inside forms were built in small sections and were supported on trusses which were

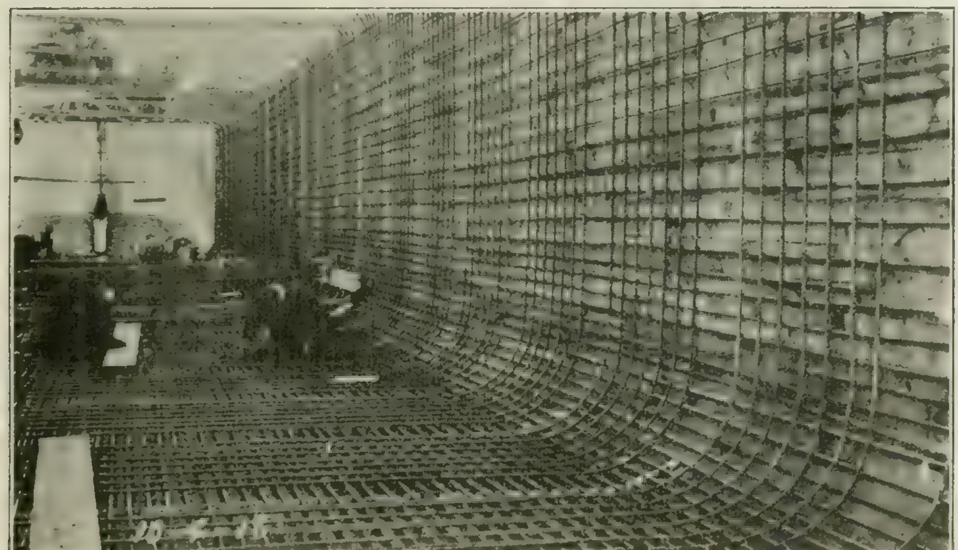
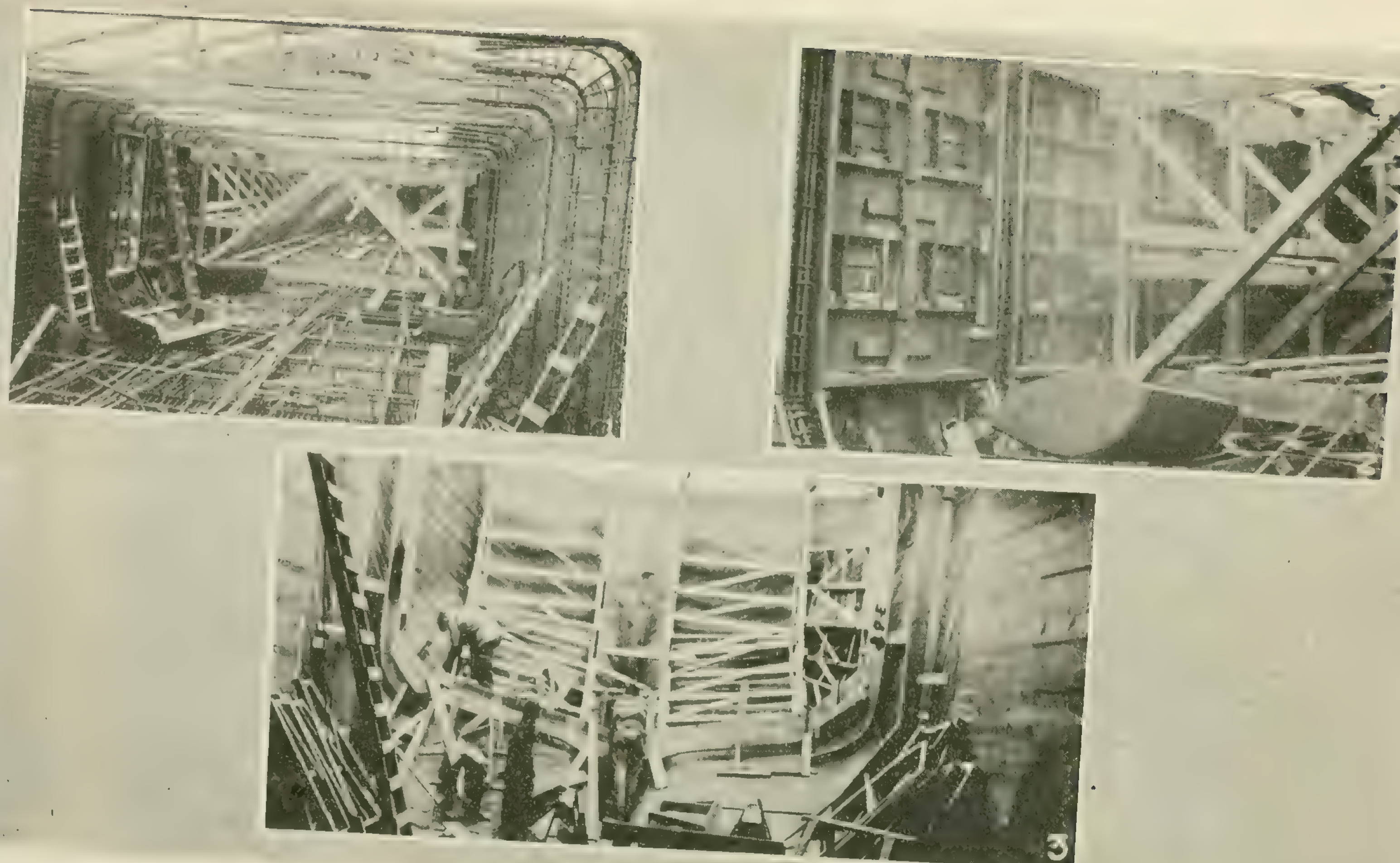


FIG. 12.—TWO LAYERS SHELL REINFORCEMENT PLACED.

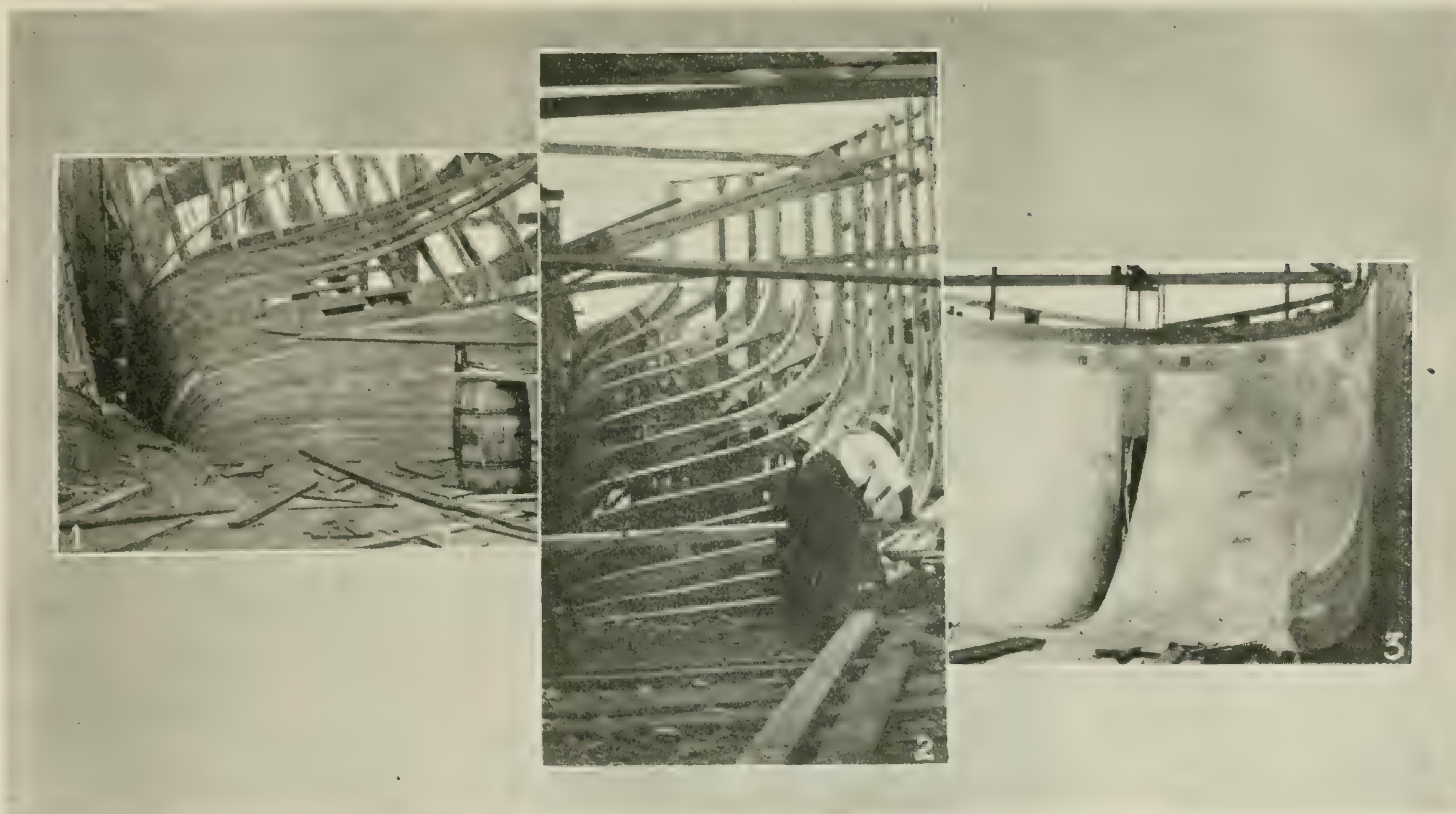
carried overhead between vertical supports of the outside forms. The inside face of the shell of the barge was formed by small panels about 16 in. high and extending from one transverse rib to the next, well braced by a framework of 2 by 4's.

The yard at Detroit, Michigan, operated by Thomas E. Currie, consisted of three graving docks excavated to a depth of 9 ft. and connected to a lagoon leading to the Detroit River. Each dock was provided with a concrete floor which formed the bottom form of the shell of the barge. The outside forms or the parallel portion of the barge consisted of $\frac{7}{8}$-in. sheathing run continuously along the barge and supported by reinforced-concrete jacks spaced 5 ft. on centers. These jacks rested on long tapering wedges, which, when removed, would allow the jacks to tip outward, thus freeing the hull of the outside form. The curved portion of each side of the bow was formed by sheathing carried on a single unit of framework. The outside form for the curved portion



1. Inside Forms Being Placed, Ithaca. 2. Windows for Concreting Shell, Ithaca. 3. Framing Inside Stern Forms, Tonawanda.

FIG. 13. VIEWS OF THE INSIDE FORMS FOR BARGES.



1. Tonawanda. 2. Fort Edward. 3. Detroit.

FIG. 14.—METHODS OF CONSTRUCTING OUTSIDE STERN FORMS.

of the stern was constructed of cement mortar plastered to a thickness of about 2 in. upon extended metal lath. This cement form, as well as the concrete bottom floor, was coated with a mixture of plumbago and tallow to prevent adhesion at the time of concreting the barge.

The inside forms were made in sections which, when assembled, were carried upon the reinforcing steel through the bearing obtained by the pipe-sleeve inserts in the transverse beams in the bottom of the barge. The inside face of the side shell was formed by platens of $\frac{7}{8}$-in. sheathing extending from one rib to the next and about 18 in. high. These platens were dropped in turn as the concrete was deposited behind them.

The plant layout included a woodworking shop, an office, a blacksmith shop, a bending shop, and a floating concrete plant.

FIG. 15.—SHEATHING OUTSIDE STERN FORMS AT ITHACA.

The floating concrete plant consisted of a steam-driven one-yard cube mixer mounted improvisedly at one end of a floating dredge. To this dredge was attached a barge having the materials bins which were equipped with belt conveyors. The concrete was elevated in a tub and deposited in a hopper, from whence it was carried by a traveling bucket on a gantry crane which spanned the berth in which the barge being concreted was located. By means of cables the whole floating concrete plant, with gantry attached, could be moved backward and forward in the lagoon alongside of the barge under construction, so that concrete could be deposited in any place desired.

Launching was accomplished by opening floodgates from the lagoon into the basin and at the same time applying hydraulic pressure to a row of chambers located in the concrete floor of the basin immediately under the keel of the barge. It was found in most cases that these hydraulic chambers

were not needed to raise the barge off the floor of the basin. After flotation, docking blocks were set and the water pumped out of the basin, thus leaving the barge in a dry dock for examination of the outer surface of the bottom and for accessibility for applying the exterior coating.

The concrete used in the construction of the barges was mixed in the proportions of one part cement, two-thirds parts sand under $\frac{1}{8}$ in., and one and one-third parts gravel between $\frac{1}{8}$ in. and $\frac{1}{4}$ in. The average consistency used was one giving $8\frac{1}{2}$ in. slump from a 6 x 12-in. cylinder mold, when the mold, previously filled with fresh concrete, was drawn upward allowing the mixture

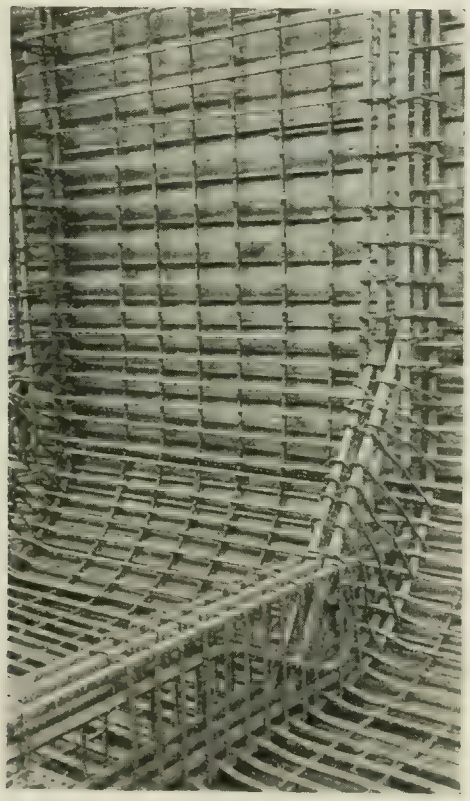


FIG. 16.—STEEL IN FRAME AND SHELL.

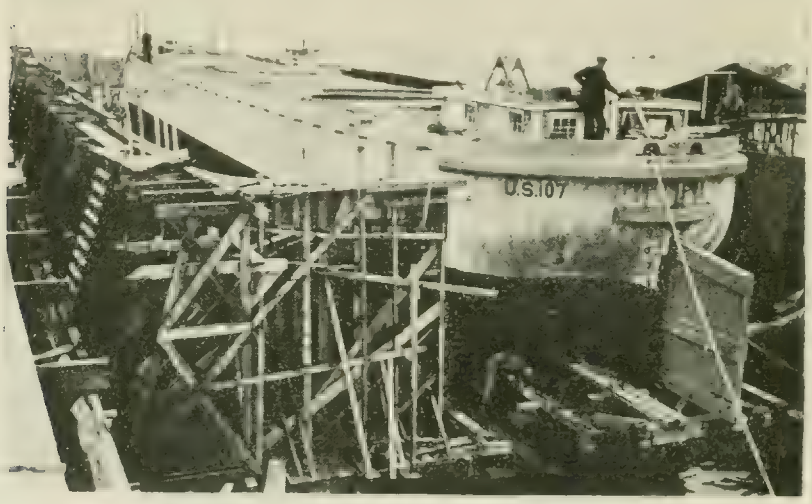
to settle and spread. The aggregate was in all cases obtained in the vicinity of the yard. The concrete produced has shown a compressive strength in 28 days of from 3500 to 4500 lb. per sq. in.

Probably the most interesting features concerning the construction of these barges are the methods of concreting. Not only was a richer mix employed than is common, but a dryer mix as well, necessitating the use of vibrating hammers on the forms in order to cause the mixture to flow into the large mass of reinforcement.

In the yard at Detroit the concrete was conveyed from a floating plant into the barge by means of a bucket with double hopper bottom operating on a gantry crane. All of the concrete for pouring the barge was delivered

to either of two levels: an intermediate working platform or the deck. The intermediate working platform was about $6\frac{1}{2}$ ft. above the bottom of the boat. All concrete placed below the deck level was placed from the working platform. Only the concreting of the deck was performed from the deck itself.

The first barge built in this yard was poured with a somewhat different



Ready to Launch, Fort Edward.

Covered for Winter Work, Ithaca.

FIG. 17.—VIEWS OF SOME OF THE BARGES.

distributing system than that used in the later barges. For the bottom and the bilges, the concrete was lowered to temporary runways laid on top of the keelsons and frames. After placing the concrete in the side walls for one lift above the bilge, this temporary floor was removed and the concrete for the remainder of the side walls was handled from the working platform. Because

of the inconvenience of finishing the bottom with this planking lying on the frames, and because of the débris scattered over the finishing surfaces while removing this temporary platform, it was thought best to handle all concrete below the working platform from this platform.

By means of a bucket suspended from the gantry, concrete was lowered into the barge in every other frame bay. This bucket was provided with two hoppers in its bottom, from which concrete could be discharged in either of the two lines of buggies operating longitudinally on the working platform. After being filled with concrete, these buggies were moved backward or forward to the place where the workmen were placing concrete into the forms. At this point, the concrete was shoveled from the buggy into a metal sectional chute which led directly to the place in the form that was being filled. Various

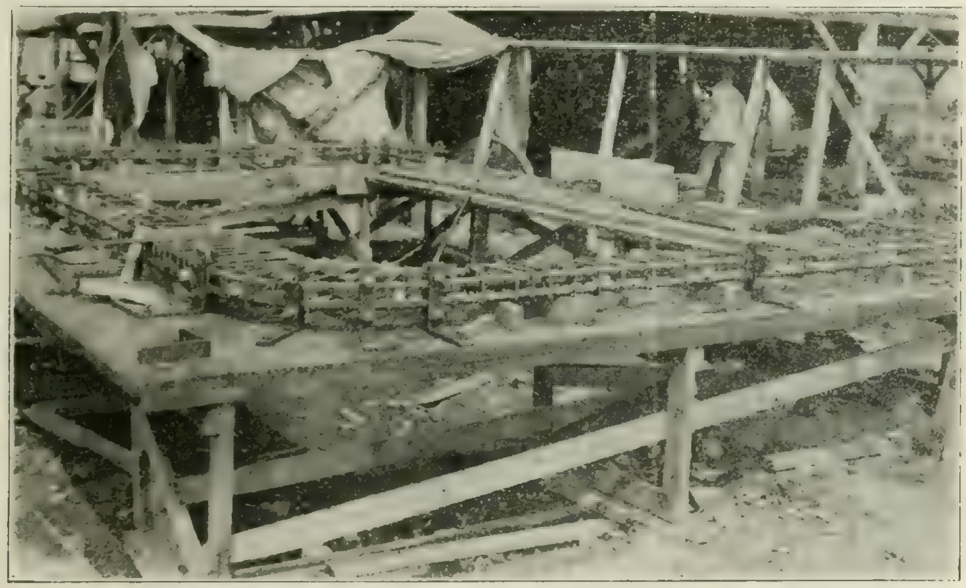


FIG. 18.—MAIN FRAME STEEL FABRICATED ON TABLE, DETROIT.

types of light sheet metal aprons and hoppers at the forms have been used with great success for receiving the concrete.

At Fort Edward the first method employed for conveying the concrete was by chuting. For this purpose it was necessary to construct a tower about 150 ft. high. The spout from the top of this tower made a slope with the horizontal of about 50 degrees. It discharged into a mixing box at about the center of a runway, running longitudinally along the center line of the boat and supported by transverse trusses at an elevation of about $3\frac{1}{2}$ ft. above the deck. From this mixing box the concrete was wheeled toward either end of the barge in wheelbarrows, being then dumped into the hopper-shaped tops of chutes extending into the interior of the barge and discharging into a small portable mixing box, from which the workmen shovel the concrete directly into the openings of the forms.

The use of the spout was discontinued after pouring the first barge because

of the difficulty in keeping the heavy mix employed from choking the spout and causing frequent cleaning. The plan now employed is to elevate the concrete in the same tower to a point slightly above the highest point of the barge as it stands on the ways. From this elevation on the tower, the concrete is wheeled in Ransome carts along a runway which is inclined slightly downward toward the barge being concreted. Since the barges in this yard are launched endwise, wheeling of all concrete is, therefore, on a down grade.

In the Ithaca yard there were two ways constructed for launching the barges sidewise. In the space between the barges, as they stood end to end on the ways, was constructed an elevating tower with a platform leading to and extending over the deck of each barge. This platform was provided at intervals with small trap doors, which served as inlets to down-chutes into the

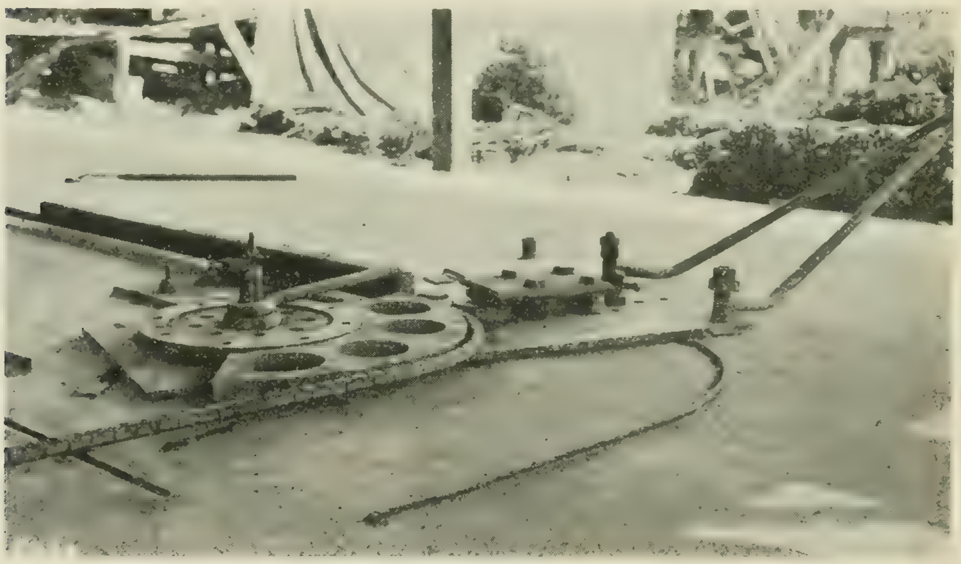


FIG. 19.—BENDING BARS AT THE DETROIT YARD.

interior of the barge. Ransome buggies were filled at the mixer, raised by the elevator to the runway at the deck level and wheeled along the runway to the trap door through which concrete was desired to be dumped.

A secondary local distributing system was employed in the interior of the barge. The scheme consisted of small boxes into which the concrete was discharged, which were slid along tracks formed of 2 by 4's. Two of these tracks were supported on transverse bracing about two feet above the tops of the keelsons. Two other lines of track were supported on other bracing at an elevation of about one-half way between the bottom and the deck. These tracks were somewhat closer to the side wall than the tracks on the lower level. Each of these tracks was served by inclined chutes which led downward from the trapdoors in the runway overhead. The boxes were shoved along these tracks to the point where the workmen were shoveling the concrete into the openings in the side forms or bottom forms, as the case

might be. Small metal hopper-shaped aprons were provided at the little windows cut in the side forms, as described above.

The barges in the yard at Tonawanda were built in a graving dock. The dock was formed by an excavation about 8 ft. below the natural ground level. The barges were constructed on four ways lying end to end. The mixing plant was located on the natural ground elevation alongside the excavation. Concrete was discharged from either of two small mixers into Ransome buggies which were drawn by a cableway up an incline to a runway along the near side of the barge at the deck level. From this runway there extended, at frequent intervals, small platforms leading into the hatch coaming, upon

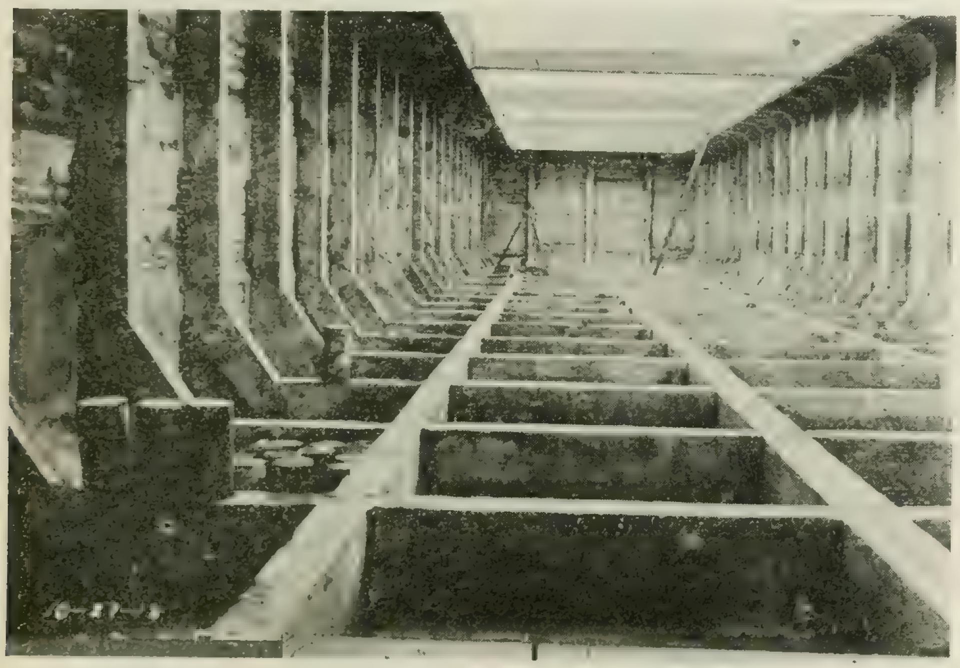


FIG. 20.—VIEW INSIDE OF THE BARGES AFTER CONCRETING.

which the buggies were wheeled and dumped over the hatch coaming into chutes leading into the interior of the barge.

Arranged similar to the working platform in the Detroit yard, there was a platform about 6 ft. above the bottom, extending fore and aft near each side of the barge. Small two-wheeled factory dollies with caster wheels on the front and back, having mortar boxes mounted upon them, were wheeled backward and forward along these platforms from the down-chutes to the location where the concrete was desired by the men placing it in the forms. All concrete placed below the deck level was handled from this working platform. Small wooden chutes were employed to spout the concrete from the working platform to any opening in the forms below the platform level.

The progressive development brought about by pouring the eighteen barges now concreted has resulted in a definite order of operations, which may be tabulated as follows:

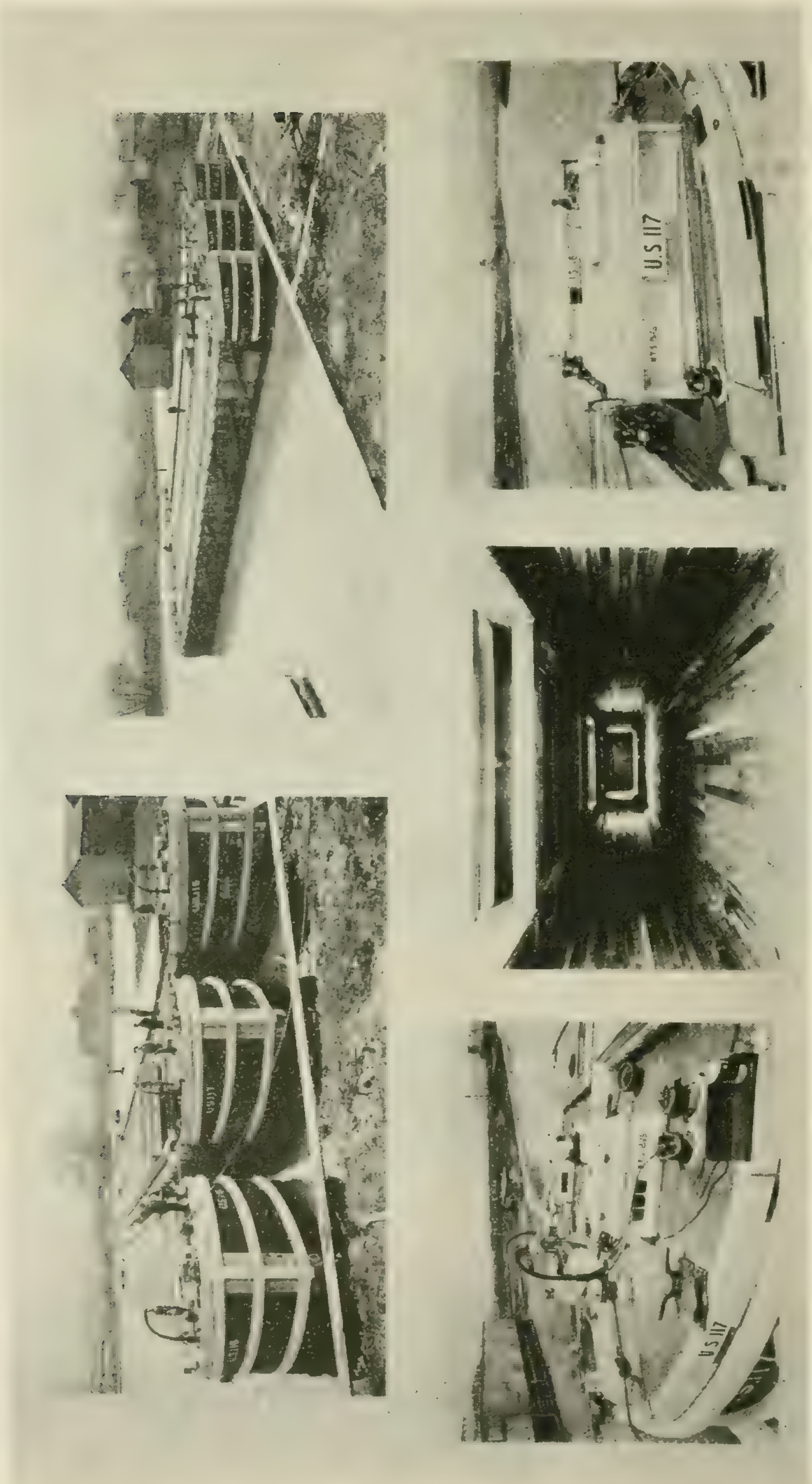


FIG. 21.—SOME VIEWS OF THE FINISHED CONCRETE BARGES FOR NEW YORK CANALS, ITHACA YARD.

Concreting Bottom and Bilges:

1st—Pour the bilges.
2d—Pour the frames, beginning at the bilges and working toward the center line of the vessel.
3d—Pour the keelsons.
4th—Pour the bottom slab.

Concrete the Side Walls:

Pour the side walls in separate lifts not to exceed 30 in. in height, the intervals between lifts to be preferably about four hours.

Concreting the Deck:

1st—Pour the deck beams including the peripheral beam.
2d—Pour the deck slab.

If the barge is poured in one operation the time of pouring which has given in general the best results has ranged from 40 to 50 hours, unless special precaution is used to prevent the forms of the side wall from spreading during concreting.

If the pouring is done in three operations on three successive days divided respectively into (1) bottom and bilges, (2) side walls, and (3) gunwale and deck, which system is used at the present time at the Fort Edward yard due to shortage of labor, the actual pouring time has averaged about one-half of the time required for pouring in a single operation.

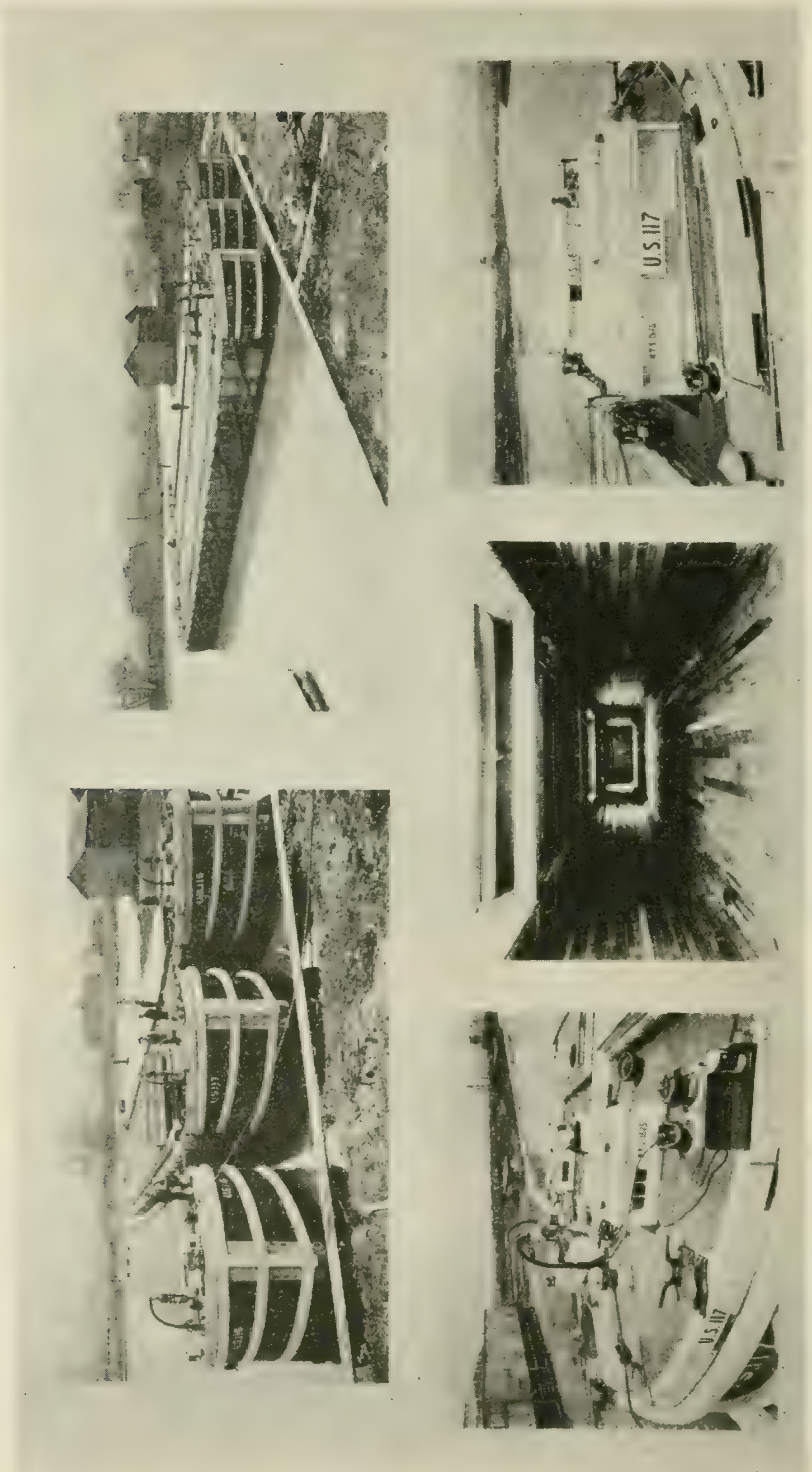


FIG. 21.—SOME VIEWS OF THE FINISHED CONCRETE BARGES FOR NEW YORK CANALS, ITHACA YARD.

Concreting Bottom and Bilges:

1st—Pour the bilges.
2d—Pour the frames, beginning at the bilges and working toward the center line of the vessel.
3d—Pour the keelsons.
4th—Pour the bottom slab.

Concrete the Side Walls:

Pour the side walls in separate lifts not to exceed 30 in. in height, the intervals between lifts to be preferably about four hours.

Concreting the Deck:

1st—Pour the deck beams including the peripheral beam.
2d—Pour the deck slab.

If the barge is poured in one operation the time of pouring which has given in general the best results has ranged from 40 to 50 hours, unless special precaution is used to prevent the forms of the side wall from spreading during concreting.

If the pouring is done in three operations on three successive days divided respectively into (1) bottom and bilges, (2) side walls, and (3) gunwale and deck, which system is used at the present time at the Fort Edward yard due to shortage of labor, the actual pouring time has averaged about one-half of the time required for pouring in a single operation.

REPORT OF THE COMMITTEE ON CONCRETE SHIPS.

The committee has no report except the following statement from Mr. Robert W. Boyd, Assistant Head of the Concrete Ship Division of the Emergency Fleet Corporation, as to work that has been done in this country in the construction of barges and ships. It seemed to the committee that it might be wise to make this record in our Proceedings.

It seems to the committee that this indicates a tremendous development of the concrete ship in the past two years. As to the future of the concrete ship, the Chairman is not sufficiently in touch with the construction of concrete ships to make any forecast, and I find few people who are willing to make a forecast at this time of the commercial future of the concrete ship. Apparently, from the standpoint of scientific design and construction, it seems entirely feasible to build concrete barges and ships, but as to their commercial value in competition with steel, the opinion seems to be that this must remain a problem to be decided after the experience gained from the boats launched and the boats which are under construction.

The statement follows:

In Service:

Five 400 to 700-ton tank barges, built by the Fougner Concrete Shipbuilding Co. for the Standard Oil Co., New York.

One 1,200-ton deck barge, built by the Fougner Concrete Shipbuilding Co. for the Standard Oil Co., New York.

One 500-ton deck barge, built by the Concrete Boat Co., San Francisco.

One 500-ton deck barge, built by L. L. Brown Co., Peekskill, N. Y.

Two 500-ton barges, built by the Aberthaw Construction Co., Boston, Mass.

One 500-ton barge, built by the L. B. Harrison Co., Long Beach, N. Y.

Two 500-ton deck barges, built by the Arundell Sand and Gravel Co., Baltimore, Md.

Two 500-ton deck barges, built by the Thompson Sand and Gravel Co., Mobile, Ala.

Two 500-ton barges, built by the Inter-Ocean Barge and Transport Co., Seattle, Wash.

Three 500-ton canal barges, built by the Holler-Davis-Flood Co., Fort Edward, N. Y., for the United States Railroad Administration.

Five 500-ton canal barges, built by Thomas E. Currie, Detroit, Mich., for the United States Railroad Administration.

Four 500-ton canal barges, built by the Cummings Structural Concrete Co., Ithaca, N. Y., for the United States Railroad Administration.

Three 100-ft. water tankers, built by the Great Northern Shipbuilding Co., Portland, Ore., for the War Transport Service, War Department.

Two 500-ton harbor lighters, built by the Amberson Construction Co., Little Ferry, N. J., for the Navy Department.

Three 500-ton harbor lighters, built by the L. L. Brown Co., Peekskill, N. Y., for the Navy Department.

Launched, But Not in Service:

7,500-ton tanker "Palo Alto," built by the San Francisco Shipbuilding Co., Oakland, Cal., for the Emergency Fleet Corporation.

3,500-ton cargo ship "Polias," built by the Fougner Concrete Shipbuilding Co., Flushing, N. Y., for the Emergency Fleet Corporation.

3,000-ton cargo ship "Atlantus," built by the Liberty Shipbuilding Co., Brunswick, Ga., for the Emergency Fleet Corporation.

Six 225-ft. car floats, built by the Liberty Shipbuilding and Transportation Co., Cleveland, Ohio, for the War Transport Service, War Department.

One 225-ft. car float, built by L. B. Harrison Co., Athens, N. Y., for the War Transport Service, War Department.

Under Construction:

Two 7,500-ton tankers, by the Pacific Marine and Construction Co., San Diego, Cal., for the E. F. C.

One 7,500-ton tanker, by the San Francisco Shipbuilding Co., Oakland, Cal., for the E. F. C.

One 7,500-ton cargo ship, San Francisco Shipbuilding Co., Oakland, Cal., for the E. F. C.

Two 7,500-ton tankers, by the Fred T. Ley Co., Mobile, Ala., for the E. F. C.

One 7,500-ton cargo ship, by the Fred T. Ley Co., Mobile, Ala., for the E. F. C.

Two 7,500-ton tankers, by the A. Bentley & Sons Co., Jacksonville, Fla., for the E. F. C.

Two 3,500-ton cargo ships, by the Liberty Shipbuilding Co., Wilmington, N. C., for the E. F. C.

Five 500-ton canal barges, by the Holler-Davis-Flood Co., Fort Edward, N. Y., for the United States Railroad Administration.

Two 2,000-ton oil tankers, for France and Canada Oil Transport Co., Aransas Pass, Texas.

Four 500-ton canal barges, by the Caldwell-Marshall Co., Tonawanda, N. Y., for the United States Railroad Administration.

H. C. TURNER, *Chairman.*

REPORT OF THE COMMITTEE ON FIREPROOFING.

In the interval since the last report of this committee was made, a series of fire tests of more than one hundred full-size building columns, including all the important types of fire-resistive columns, has been conducted, jointly, by the National Board of Fire Underwriters, The Associated Factory Mutual Fire Insurance Companies and the Bureau of Standards. These tests will give a vast amount of much-needed information as to the relative efficiency and reliability of the different types of protection for steel columns and will also give important information as to the fire-resistive properties of several types of concrete columns. The report of the results of these tests has not been completed and it would be premature to undertake to draw on this storehouse of information for this committee's report for this year. It will undoubtedly be available for next year's report.

Supplementing the tests made at Underwriters' Laboratories, the Bureau of Standards has conducted a series of fire tests of over fifty full-size columns, all of concrete. The results of a large part of this work were presented in a report to the Institute at its meeting last year and the results of the remainder of the tests are available for use at this time. The information furnished by the reports on the Far Rockaway Fire by the National Board of Fire Underwriters and a special committee of the American Concrete Institute contain information that should be given consideration in the report of this committee as should important reports of the British Fire Prevention Committee on two large and disastrous factory building fires which occurred in London in 1917. Certain important conclusions have been drawn from the information available from these and other sources. It is the purpose of this report to present these conclusions, together with definite engineering recommendations based thereon.

Thickness of Protective Material.

The Bureau of Standards' column tests show, quantitatively, the relative effects of a standard 4-hr. fire test on various types of columns made with concrete from various aggregates. A thickness of 1½ in. of concrete was provided, outside of the steel, in all cases. The steel was probably more accurately placed, in the forms, than could ordinarily be expected in actual building operations. In most of the 18-in. round columns, temperatures between 550 and 600 deg. C. (1022 and 1112 deg. F.) were reached in the steel at the end of the 4-hr. fire test. In the 16-in. square columns, temperatures in the steel were above 600 deg. C. (1112 deg. F.). Steel at these temperatures has lost a large part of its strength. It is evident, therefore, that 1½ in. of protective concrete is not sufficient thickness for the proper protection of the steel where 4-hr. protection is required.

In three columns in which the protection had been increased to 2½ in. by the addition of cement mortar in the form of plaster, with light mesh

reinforcement, the temperatures in the steel at the end of the test were approximately 400 deg. C. (752 deg. F). It is, therefore, recommended that in concrete columns where 4-hr. protection is required, protective material not less than 2 in. in thickness be provided, over the steel. In columns in which a high percentage of steel is used, increasing the importance of affording it ample protection, the thickness of protective material should be 2½ in. for 4-hr. protection, and special care should be given to the accurate placing of the steel in the forms, to avoid inadequate protection on one side.

AGGREGATE.

Previous fire tests have indicated that there is an important difference in the fire-resistive properties of different aggregates. Ira H. Woolson, in his final report to the American Society for Testing Materials of investigations of the fire resistive properties of concrete made in 1905, 1906 and 1907, reported adversely on the pure quartz gravel which is extensively used in the vicinity of New York City. Shortly afterward, the British Fire Prevention Committee reported adversely on the behavior of gravel concrete in a fire test of floor arches. Further evidence was developed by tests made by the Bureau of Standards and reported to the American Society for Testing Materials in 1917. These test results have been so consistent that it appears to have been well established that the physical properties of gravel, that is high in quartz content, are not such as to make it as suitable as other aggregates for fire-resistive concrete.

Investigation of the failure of concrete construction in recent severe fires clearly indicates that the damage was much greater than would have been the case if any one of a number of aggregates, other than gravel of high quartz content had been used. This was especially true in the case of the fire at Far Rockaway, New York. This fire has been thoroughly investigated. A complete report of the fire and of the condition of the building, with conclusions attributing the damage in large part to the pure quartz gravel aggregate, was prepared for the National Board of Fire Underwriters by its Consulting Engineer, Ira H. Woolson. Copies of this report are obtainable. A further investigation was made by a special committee of the American Concrete Institute and a report presented by Richard L. Humphrey, dealing with the fire, the condition of the building and the methods used in reconstruction. Great importance was attached to the nature of the aggregate in this report.

To this evidence may be added that of two great factory building fires which occurred in London in 1917. Both of these latter fires were of great severity and long duration. Both were started by an explosion. It was impossible to determine how much of the damage was caused directly by the explosion, but after thorough investigation the British Fire Prevention Committee attributed a large part of the damage to the inability of the Thames ballast (gravel) concrete to withstand the destructive stresses of a severe fire, as is shown by their reports, from one of which the following two paragraphs may be quoted:

"It is clearly shown that Thames ballast or gravel concrete is an unsuitable material for reinforced concrete in buildings where the risk of a severe fire has to be considered.

"Where a suitable aggregate or combination of materials is used in the future and the reinforcement accorded proper protection, only superficial damage need be expected, *i. e.*, damage repairable without extensive reconstruction."

In the fire tests of concrete columns made by the Bureau of Standards, gravel concrete columns were shown to be greatly inferior in fire resistance to columns having other aggregates. While this applies to all types of columns, the contrasts were greatest in round columns, vertically and spirally reinforced and in vertically reinforced square columns. It is probable that similar results would be obtained from all gravels made up largely of highly silicious materials such as sandstone pebbles, quartz pebbles and granite pebbles.

It is clearly indicated that serious structural damage by fire in concrete buildings could be prevented in large measure by using, where it is economical to do so, either limestone, trap rock, blast furnace slag or burned clay aggregate and, where none of these aggregates can be used economically, by paying special attention to certain structural details. In the case of columns, there is evidence that vertically reinforced round columns, without spiral, resist fire better than square columns without spiral. This applies particularly to gravel concrete columns. It does not appear to be true of columns made from the more suitable aggregates. Gravel concrete columns having spiral reinforcement appear to be particularly susceptible to spalling. It has been found that the addition of 1 in. of cement plaster, held in place by light metal mesh, was sufficient to prevent the loss of the protective covering by spalling in the case of spirally reinforced gravel concrete columns, and that columns so protected lost but little of their original strength in the 4-hr. test. There seems to be added merit in such protection in the fact that the thickness is uniform over the columns, whereas any inaccuracy in placing steel would result in protective concrete being thinner on one side than on the other.

The following recommendations are therefore made, pending further developments along this line:

1. That for fire-resistive construction, limestone, trap rock, blast furnace slag and hard burned clay be given a preference over highly silicious gravels.

2. That in cases where gravel aggregate is to be used, with no additional protective material over the concrete, round columns be given a preference over rectangular ones.

3. That where gravel aggregate is used, all columns, but especially rectangular columns and round columns with spiral reinforcement, be given the additional protection of approximately one inch of portland cement plaster either on metal lath or reinforced by light expanded metal.

WALTER A. HULL,
Chairman.

DISCUSSION.

Presented by the Chairman of the Committee, President W. K. Hatt in the Chair.

Mr. Rudolph Hering.—In reference to the expression "burned clay," I had some experience in England with the material which receives that name there. It is quite a specific material; I had never seen it before and I have not seen it anywhere else. It is produced by breaking clay into irregular masses and burning it in such a way that the pieces are quite jagged and are very hard. It is considered an excellent material for sewage purification. Another thing I saw in Liverpool was a number of specimens of concrete broken so as to show type of fracture. Concrete made of limestone and granite and other similar material broke with rough surfaces, but the concrete made with gravel broke on the smooth surfaces of the gravel. Generally the cement separated from the smooth surfaces of the gravel, whereas, in the other cases, there were about half of the pieces of aggregate that had been broken through. The cement, therefore, was stronger than the stone, and at that time they told me "you must not have smooth surfaces to your aggregate if you want strong concrete." **Mr. Hering.**

Mr. I. H. Woolson.—In reference to what the last speaker has said, my own experience does not bear out that conclusion. In the Far Rockaway fire, that the paper has referred to, and on which I made a report, I found that concrete in the beams and girders, which failed owing to the extension of the reinforcement bars, had been overheated in the bottom of the beams or girders and much weakened. The concrete in the upper part of the beams and girders, where crushed by compression, often broke completely through the pebbles, and the mortar was attached solidly to the pebbles, so that it could scarcely be chipped off with a hammer. Where it had been submitted to heat there was very marked cleavage between the surface of the pebbles and the attachment of the mortar, which produced disintegration. I think that good concrete, strong enough for all practical purposes, can be made from gravel where it is not liable to be submitted to heat. **Mr. Woolson.**

Mr. D. E. Douty.—I notice that the committee places limestone as one of the preferential materials. If my memory serves me right, the experiments which Professor Woolson reported to the American Society for Testing Materials in 1907 to 1909, or thereabouts, and also the slabs which were tested at the Underwriters' Laboratory at about the same time, and the investigations which Mr. Humphrey conducted at the San Francisco fire, showed that limestone aggregate was not a very protective coating. I may be wrong about that, but it seems to me that those earlier results showed that. Perhaps Professor Woolson will confirm it. **Mr. Douty.**

Mr. Woolson.

MR. WOOLSON.—All of us make statements some times that we wish we had not made. I have made some that I wish I had not. I went on record as saying that limestone was not a suitable aggregate for concrete that might be subjected to fire. Mr. Hull has proved conclusively that I did not know what I was talking about. I admit it frankly.

I drew my conclusions principally from a single test. Mr. Douty has called attention to some investigative work which I did in 1905 to 1907 upon the effect of heat upon concrete, in which some of the specimens were limestone. I do not remember exactly how the strength of those limestone specimens stood up as compared with trap rock; I know they were better than the gravel. The gravel was very much inferior, and that was where I got the first clue that gravel concrete was not suitable to resist fire. Later I happened accidentally to make a comparative fire test of trap-rock and limestone concrete in a wall. It was a 4-hr. fire test on a 4-in. partition wall in which about 3 ft. of the middle portion was of limestone concrete and the balance was trap-rock concrete. At the expiration of the fire test, water was applied from a hose in the usual manner. The effect was to wash away the calcined limestone and the surrounding mortar ½ in. to 1 in. in depth, whereas the trap-rock concrete was still in perfect condition and showed the markings of the grain of the wood in the form boards. On almost that single test I went on record that limestone was not a suitable aggregate. Now, as a matter of fact, so far as the fire was concerned, that limestone did perform its function satisfactorily up to the time that water was applied; the calcined limestone itself was an excellent retardant of heat, and although the water did wash away the calcined material as described, the damage was not as serious as might appear. I had one other experience with limestone concrete under fire and water test and was surprised at the structural strength developed after the test. Not having measured the interior temperature of the concrete I did not realize the heat-resisting properties of the concrete, and judged it by the effect of water on the calcined stone.

Mr. Hull's tests have shown that his limestone concrete columns gave excellent results and that the strength of the columns was only slightly impaired by the effect of the fire tests, so I take back all that I have said against limestone concrete, but I still consider trap rock the better aggregate to resist fire.

Mr. Humphrey.

MR. RICHARD L. HUMPHREY.—The reference Mr. Douty made to tests made in the Underwriters' Laboratory, which were the tests of the 8 x 13-in. beams, composed of cinder concrete, granite concrete, gravel concrete and limestone concrete, showed that the penetration effect of the fire on the limestone was not anything like as great as was expected. It stood up as well as some of them, although in those tests I think the results show that the gravel concrete came through with about the same minimum amount of damage. Granite, I think, showed the best test, but the tests did not prove the statement he made that the limestone had been inferior in the test.

REPORT OF COMMITTEE ON CONCRETE SEWERS.

Your Committee on Concrete Sewers submits herewith specifications entitled "Proposed Standard Specifications for Monolithic Concrete Sewers and Reinforced-Concrete Pipe Sewers and Recommended Rules for Concrete Sewer Design." These specifications are in revision of those submitted by the committee at the 1918 meeting, and in addition there has been introduced specifications for concrete pipe sewers.

The committee has carefully considered the criticisms of the specifications submitted in 1918, and the draft herewith is drawn in general to meet those criticisms. It is only fair to state, however, that a number of members of the committee were in favor of retaining many of the provisions embodied in the last draft which were subject for criticism. The committee wishes to emphasize that any specifications for sewers, of whatever material, must be based on the presumption that the masonry will have to be placed under the worst possible construction conditions. The workmanship, which can be relied upon to produce satisfactory results in the mixing and placing of concrete in buildings, or even in pavements, may not produce these results under the very unfavorable conditions which frequently exist in the sewer trenches. It is, therefore, extremely important to increase the factor of safety against defective construction in every reasonable way.

The committee has retained the provision of the 1918 specifications introducing a coefficient of hardness in the specification for fine and coarse aggregate. In introducing a coefficient of hardness in the specifications for fine and coarse aggregate in Sections 3 and 4, it was the intention of the committee to prohibit entirely the use of ordinary middle western limestones for coarse aggregate, and particularly the screenings from these limestones for fine aggregate. We note that these stones and the screenings have been used extensively in concrete work, and, to some extent, in underground hydraulic work, but both from the analysis of the material, in respect to the conditions under which the sewers must operate, and from a small amount of data from the condition of the older concrete sewers, we are of the opinion that sewers built of hard aggregates would be so much more satisfactory that the use of these materials should not be considered. We believe that the coefficient of hardness at 16, which has been set, will permit of screenings from gran tes and trap rock, as well as from the very hard limestones, but the softer stones, which are likely to be disintegrated and partially dissolved under the warm, moist conditions existing in the sewers, will be prohibited. It was suggested that "screenings have been successfully used in this class of work, or in other classes of a similar nature, without any difficulty." The committee appreciates that the limestone screenings, no matter how soft, make a workable concrete, but feels fithat it must give more consideration to the life of the structure than to the ease of construction.

The provision of the 1918 specification calling for deformed bars for reinforcement has been omitted. The committee is thoroughly familiar with tests which have shown that the deformations on the bars are not effective until incipient failure, but nevertheless some members feel that the use of deformed bars in sewer work would be a reasonable factor of safety.

The suggestion for the use of hydrated lime, incorporated in the previous specifications, is omitted. Here again some members of the committee feel that the use of hydrated lime in sewer work is well warranted, both to secure additional density, and additional workability of the concrete. All members agree that satisfactory concrete can be obtained without the lime, but some believe that its use would be highly valuable in overcoming the conditions inherent in sewer construction.

On the basis of Professor Abrams' test, the time of mix has been reduced to one minute, yet the committee is not in entire agreement on this point, and some members feel that the requirement for one and one-half minutes might result in a more workable concrete.

The committee has reviewed the specifications for granolithic invert finish which was the subject of some criticism in the last report. It finds no reason for changing this specification, but calls attention to the fact that a type of finish which would not be satisfactory on a dry floor subject to abrasion, might be exactly proper under flowing water.

The provision for a separate vitrified brick lining has been retained, and there has also been added paragraphs covering the use of clay tile liners, and concrete block liners. Some members of the committee feel that under adverse conditions a separate lining of some type is highly advisable.

Minor changes have also been made in the Recommended Rules for Sewer Design. We feel that the specifications have been considerably improved in the last revision, and that the specifications for concrete pipe sewers are very valuable. The committee recommends presentation of the specifications as a new standard of the Society.

Respectfully submitted,

W. W. HORNER, *Chairman.*
ARTHUR S. BENT,
ALFRED H. HARTMAN,
W. S. LEA,
FRANK A. MARSTON,
LANGDON PEARSE,
COLEMAN MERIWETHER,
J. L. ZEIDLER,
W. R. HARRIS.

[The report as presented on the floor is printed on the succeeding pages and is followed by the discussion, with amendments from the floor, and the reply of the committee.—EDITOR.]

PROPOSED STANDARD SPECIFICATIONS FOR MONOLITHIC CONCRETE SEWERS AND REINFORCED-CONCRETE PIPE SEWERS AND RECOMMENDED RULES FOR CONCRETE SEWER DESIGN.

REVISED 1919.

PART I.—MATERIALS.

Section 1. The materials required for the construction shall be furnished by the contractor and will be inspected by the engineer. Defective materials shall be removed from the site of the work and defective work repaired or replaced as directed. Facilities for the handling and inspection of materials and work at all times shall be furnished by the contractor.

If the work, or any part thereof, shall be found defective at any time before the final acceptance of the whole work, the contractor shall forthwith make good such defects in a manner satisfactory to the engineer.

Cement.

Section 2. All cement shall conform to the current specifications for portland cement of the American Society for Testing Materials, and shall be tested in accordance with the methods of testing described in the specifications of that Society.

Fine Aggregate.

Section 3. Fine aggregate shall consist of sand graded from fine to coarse and passing when dry, a screen having holes one-quarter inch in diameter. It shall be clean, coarse, free from dirt, vegetable loam or other deleterious matter. Not more than 6 per cent shall pass a sieve having one hundred mesh per lineal inch.*

Section 4. Fine aggregate shall be of such quality that mortars composed of the proportions of cement and fine aggregate hereinafter specified for the various classes of concrete shall show a compressive strength after fourteen (14) days at least equal to the strength of mortar made of portland cement and standard Ottawa sand in corresponding proportions and of the same consistency.†

Coarse Aggregate.

Section 5. The coarse aggregate shall consist of crushed stone or gravel which is retained on a screen having ¼ in. diameter holes and graded from the smallest to the largest particles. It shall be clean, hard, durable, free

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* Crushed stone screenings may be permitted for use as fine aggregate provided that they shall comply with all the specifications of Sections 3 and 4, and further that they shall be produced from stone having a coefficient of hardness of not less than 16, as described in Bulletins No. 347 and No. 370 of United States Department of Agriculture.

† It is recommended that, if possible, available fine aggregates be tested before awarding contracts. If it appears necessary to use an aggregate of poorer quality than above specified, the proportion of cement in the various classes of concrete should be increased in order that the strength of the mortar actually used shall not be less than that with the Ottawa sand. If, however, the strength of the resulting mortars is less than 70 per cent of those with Ottawa sand, fine aggregate should be rejected entirely.

from all deleterious matter, and soft, flat or elongated particles. Crusher dust in sufficient quantity to weaken the concrete will not be permitted. For reinforced-concrete arches or for plain concrete arches less than 6 in. in thickness, the maximum size of particles shall be such as will pass a screen having 1 in. diameter holes. For inverts and plain concrete arches over 6 in. in thickness, the maximum size of particles shall be such as will pass a screen having 1½ in. diameter holes.

Section 6. Where crushed stone is used it shall have a coefficient of hardness of not less than 16, "as described in Bulletins No. 347 and No. 370 of United States Department of Agriculture."

(The above paragraph is for use in locations where limestone or sandstone of a questionable value are common. If all available stone is suitable, the paragraph may be omitted.)

Samples of Aggregate.

Section 7. Samples of fine and coarse aggregates which the contractor proposes to use shall be submitted to the engineer, if so required by him, for examination at least two weeks before the contractor commences to deliver the materials at the site of the work. Materials shall not be delivered until the samples shall have been approved, and as delivered they shall in all respects be equal to the approved samples. Samples of not less than ½ cu. ft. of fine aggregate and not less than one cubic foot of coarse aggregate shall be delivered in suitable boxes or containers. All samples shall be plainly and neatly labeled with the places where taken, where to be used, the date and the name of the collector.

Section 8. For the purpose of determining proportions of materials for concrete, each bag of cement containing 94 lb., shall be considered as containing one cubic foot. Sand and coarse aggregate shall be measured loose in approved boxes or hoppers.

Water.

Section 9. Water shall be provided at the site of the work by the contractor, who shall pay for all connections to existing mains where available, and for all necessary piping along the work. Water shall be free from oil, acid, alkalies or organic matter.

Concrete Reinforcement Bars.

Section 10. All steel reinforcement shall consist of cold drawn steel wire fabric, having an elastic limit of not less than 55,000 lb. per sq. in., or of reinforcing bars.

10.1. Steel bars for reinforced-concrete sewers shall conform to the current specifications of the American Society for Testing Materials for (A) Billet Steel or (B) Rail Steel, except that rail steel bars may be used in sizes of 1 in. and under only, and hot twisted bars will not be permitted.

Section 11. Dimensions of bars given on the plans are based on square sections. The net area and weights of bars used shall not be less than 95

per cent of the values for square bars as indicated. In computing the weights of steel, one cubic inch of steel shall be regarded as 0.283 lb.

Section 12. The quantity of metal to be paid for shall be the number of pounds actually placed, as shown on the drawings or as ordered. It shall not include any waste metal due either to the nature of the construction or to the fact that the lengths supplied are too long or too short for their purpose. **Measurement and Payment.**

The quantity paid for shall, however, include extra metal in laps, where authorized, due to the fact that a single bar would be unreasonably long.

All bars shall be of the length ordered and shall be in one piece where required up to 30 ft. in length.

Should the contractor be permitted to use shorter bars than directed, he shall provide the required lap and bear the expense of the extra steel and labor that is required.

The compensations shall cover the cost of furnishing and delivering metal, including any royalty, the cutting, bending, placing, fastening in position, coating with cement and all other work and materials connected therewith.

Castings.

Section 13. The contractor shall furnish and place circular cast iron frames and covers for manholes and catch basins and any other iron castings shown on the drawings, or specified herein, necessary to complete the work. **Description.**

Section 14. All castings shall be of tough, close-grained gray iron, free from blow-holes, shrinkage, and cold-shuts. They shall be sound, smooth, clean and free from blisters and all defects. **Cast Iron.**

Section 15. All castings shall be made accurately to dimensions to be furnished and shall be planed where marked or where otherwise necessary to secure perfectly flat and true surfaces. Allowance shall be made in the patterns so that the thickness shall not be reduced. Manhole covers shall be true and shall seat at all points. **Workmanship.**

Section 16. All castings shall be thoroughly cleaned and painted before rusting begins, and before leaving the shop, with two coats of high-grade asphaltum or other suitable varnish that the engineer may direct. After the castings have been placed in a satisfactory manner, all foreign adhering substances shall be removed and the castings given two additional coats of asphaltum or other varnish as directed by the engineer. **Cleaning and Painting.**

Section 17. No casting shall be accepted, the weight of which shall be less than that computed to its dimensions by more than five per cent.

Material for Lining Inverts.

Section 18. All vitrified brick shall be uniform in size, and be not less than 8 in. by 4 in. by 2 in., nor more than 10 in. by 4½ in. by 2½ in. in length, width or thickness respectively. The brick shall be free from lime or other impurities, uniformly vitrified and annealed and shall have one

edge face such that if the brick is laid on a horizontal plane on that face, no portion thereof shall be more than 1/8 in. from the plane.

Section 19. Concrete block for sewer lining shall be uniform in size, not more than 18 in. by 12 in. in surface area and not less than 3 in. in thickness. They shall be made of Class "A" concrete, as hereinafter specified, in satisfactory molds, and thoroughly cured. They shall have an ultimate compressive strength at 28 days of not less than 2000 lb. per square inch.

Section 20. Tile liners for inverts shall not be more than 8 in. by 12 in. in surface area, and not less than 2 in. in thickness. The back of the tile shall be roughened and equipped with lugs or projections for bedding in mortar. They shall be manufactured under the general requirements covering vitrified sewer pipe, and shall comply with the standard tests of the American Society for Testing Materials for clay sewer pipe in so far as applicable.

PART II. CONCRETE FOR MONOLITHIC CONCRETE SEWERS.

Section 21. Concrete shall consist of a mixture of cement, fine and coarse aggregates and water of the qualities hereinbefore specified.

Concrete shall be of three classes proportioned as follows:

Class.	Cement.	Fine Aggregate.	Coarse Aggregate.
A	1 sack	2 cu. ft.	4 cu. ft.
B	1 "	2½ " "	5 " "
C	1 "	3 " "	6 " "

The relative proportions of fine and coarse aggregates may be modified at the direction of the engineer, provided that the proportions of cement to the total of the aggregates measured separately shall not be changed.

Mixing. *Section 22.* Concrete shall be machine mixed. The concrete mixer shall be designed to take one completed batch of materials (using whole bags of cement) and to mix that batch thoroughly before any portion of it is withdrawn from, or any portion of the succeeding batch is introduced. The mixer shall be equipped with a tank so designed that when once set it will automatically supply to the mixer the amount of water so determined. The mixer shall be equipped with an instrument for measuring the time of mix.

Section 23. Concrete shall be mixed at least one minute after all the ingredients, including water, have been discharged into the mixer. Where the character of the work will permit, concrete shall be mixed in batches of one-half to one cubic yard and the mixer speed shall not be less than 8 nor more than 15 revolutions per minute. Where small mixers are used the speed shall not exceed 18 revolutions per minute.

Section 24. No concrete shall be hand mixed except relatively small quantities and then only by special permission of the engineer.

Section 25. Where concrete is mixed by hand, the cement and fine aggregate shall be mixed dry on a properly constructed wooden or steel

platform built for the purpose until it shall have obtained an even and uniform color throughout. This mixture shall then be spread to make a bed of uniform thickness on which shall be spread the coarse aggregate and the whole wet with the required amount of water and turned with square pointed shovels at least three times or until a uniform mixture is secured, water being added from time to time if necessary. The contractor may use such other method or methods of mixing concrete by hand as the engineer may approve.

Rubble or Stone in Concrete.

Section 26. In all plain concrete, where the thickness is 15 in. or more, the contractor may embed broken pieces of sound stone, the greatest dimension of which does not exceed 6 in., and the least dimension of which is not less than three-quarters of the greatest dimension. These stones shall be set in the concrete as the layers are being rammed, in a manner satisfactory to the engineer, and so placed that each stone is completely and perfectly embedded. In general, there shall be a space of 4 in. between the stones and no stone shall come within 4 in. of any exposed face. The stone shall be thoroughly cleaned and wet before placing.

Consistency.

Section 27. In mixing concrete it is advisable to use the least possible amount of water required to obtain a workable mix, and when the aggregate is dry, 6 gal. of water to a sack of portland cement is the maximum which should be used. Where comparatively dry mix is to be used, as in inverts, and near the crown of the arches, the concrete must be thoroughly tamped until the water flushes to the surface.

Work in Freezing Weather.

Section 28. Concrete shall not be mixed nor deposited in the work in freezing weather except when directed by the engineer. If the work on concrete structures is prosecuted in cold weather, proper precautions shall be taken for removing ice and frost from the materials, including heating the water and aggregates; for protecting the newly-laid masonry from freezing, and for securing work satisfactory in all respects. Satisfactory covering for the newly-laid concrete and such additional appliances and materials as may be required therefor, including steam pipes for keeping the air warm beneath the said covering shall be provided.

Transporting and Placing Concrete.

Transporting.

Section 29. Provision shall be made for quickly transporting the concrete from the mixer to the work and with as little shaking as possible, so that the tendency of water to rise to the top may be reduced to a minimum. Any concrete which may have been compacted during transportation shall be satisfactorily remixed before being placed in the work. Any concrete delayed one-half an hour in transit shall not be used in the work and must be removed from the premises.

Placing Concrete.

Section 30. Concrete shall be deposited so as to maintain a nearly level surface and avoid flowing along the forms. It shall be continuously and sufficiently worked to expel air and to force the aggregate away from the forms. In special cases, as where concrete is deposited on slopes, a comparatively dry mixture may be used, but great care shall be exercised

to spread such concrete evenly in layers not more than 6 in. in thickness and to ram it thoroughly. In general, the methods used shall be such as to give a compact, dense and impervious concrete with a smooth surface.

Joining New Work to Old.

Section 31. For the proper bonding of new and old concrete, such provisions shall be made of steps, dovetails or other devices as may be required. Whenever new concrete is joined to old, the contact surface of the old concrete shall be thoroughly cleaned, using a stiff brush and a stream of water, if required, and shall be clean and wet at the moment the fresh concrete is placed. Where ordered, a thick wash of rich mortar shall be run over the contact surface of the old concrete. Where it is of importance that the joint between the new and old work shall be as strong and tight as possible, especial precautions shall be taken, such as picking off the top one or two inches of the old work so as to remove the laitance or washing the old cement off the surface with acid or alkali and later with water to remove all traces of them, or both, as may be required.

Finish of Concrete Surfaces.

Section 32. Special care shall be taken that all concrete surfaces shall be smooth and free from indentations or projections. All surfaces shall be free from voids, exposed stones and other imperfections. If such imperfections are found upon removing the forms, the faults shall be corrected at the contractor's expense by filling with mortar or otherwise, as directed, even to the extent of taking down and replacing unsatisfactory concrete.

Plastering of Concrete Surfaces.

Section 33. No plastering of any concrete surface shall be done unless expressly permitted and if so permitted shall be done in strict accordance with directions. No payment will be made for plastering done to correct defective work.

Masonry not to be Laid in Water

Section 34. No concrete or other masonry shall be deposited under water without permission and then only in accordance with directions. The contractor shall not, without permission, allow water to rise on any masonry until the mortar shall have set at least 12 hours.

Forms.

Forms.

Section 35. The contractor shall provide suitable collapsible centers or forms with smooth surfaces of ample strength and rigidly braced. The bracing shall be adequate to prevent deviations from the correct lines.

Section 36. The contractor shall submit the design of the forms to the engineer for approval when required, and shall have the forms erected complete in the shop where fabricated for the inspection of the engineer before shipment. All steel forms shall be neatly and accurately made with all similar parts in each longitudinal section of form interchangeable with other sections. Bent plates required to fit shall be rolled and fabricated to the correct curves before assembling. Suitable forms shall be provided for bends in the sewer. Steel filler plates shall be furnished.

Section 37. All wooden forms shall be built of clean, sound lumber, reasonably free from knots, dressed on all sides and neatly fitted. Tongued and grooved material shall be used where required. The form surface shall be watertight, securely fastened to the ribs or supports.

Section 38. No forms built up in the trench or ribs with separate pieces of wooden lagging, piece by piece, will be allowed except for specials or curves.

Section 39. No center or form shall be used which is not clean and of proper shape and strength and in every way suitable. Before placing concrete, the forms shall be coated with vaseline, form grease or other suitable substance, approved by the engineer, to prevent adherence of concrete.

Placing Reinforcement.

Work to be Done

Section 40. The contractor shall furnish and place all steel bars required for concrete reinforcement of whatever size, shape and length required, including all cutting, bending and fastening and any special work necessary to hold them accurately in place and protect them from injury or corrosion.

Placing Concrete.

Section 41. All steel reinforcement shall be placed in the exact positions and with the spacing shown on the drawings or as ordered, and it shall be so fastened in position as to prevent displacement while the concrete is being deposited.

Shaping and Splicing.

Section 42. The reinforcing steel shall be bent to the shapes shown on the drawings or as required. The ends of the bars shall be bent or hooked over if required. The length of the laps for bonding the adjacent bars shall not be less than thirty times the diameter of the bar, when the steel is designed for working stress of 12,000 lb. and not less than forty times the diameter of the bar when the steel is designed for working stress of 16,000 lb. per sq. in. Where the bars are of different sizes, the diameter of the larger bar shall be used.

Storing.

Section 43. Steel must be stored in such manner that its condition will at all times correspond to that under which the samples were taken.

PART II-A.—GENERAL CONSTRUCTION, MONOLITHIC SEWERS.

Width of Trench

*Section 44.** The width of trench for circular sewers shall be equal to the greatest outside width of the sewer. Below the springing line for such sewer, the trench shall be accurately shaped to the form of the outside of the masonry and the concrete shall have a firm bearing on the natural soil or rock at all points below the springing line.

Section 45. At the direction of the engineer, the width of the trench for sewers of he horseshoe and similar types shall be one foot greater than the outside width of masonry to allow for satisfactory bracing.

Underdrain.

Section 46. Underdrains of agricultural tile of vitrified pipe, laid in gravel or crushed stone, shall be constructed of the size, and where directed by the engineer, for the purpose of keeping the work free from water during construction, such drains to be abandoned when the work is completed; underdrains so laid shall lead to sumps or manholes, and water flowing to them shall be removed by pumping. Such pumping shall be carried on con-

* Where there is a probability that wet ground will make the shaping of circular inverts impossible, an alternate section of a suitable type shall be used.

tinuously, day and night, and the level of the ground water shall be maintained below any cement or concrete which may be placed in the work for a period of at least twelve hours after such cement or concrete is placed. When the temporary underdrains above described are abandoned, they shall be cut and plugged where directed by the engineer and the sump holes above described shall be solidly filled with approved material.

Construction of Inverts.

Section 47. On all sections having a comparatively flat invert, the contractor shall first build the complete invert while on all circular sections, he shall build a center strip which shall not be less than one-fourth circumference. The invert or center strip shall be placed in sections of not over 16 ft. where the surface is to be finished with end guides and a longitudinal straight edge, and not more than twenty feet if a separate lining of vitrified brick, tile or concrete block is to be provided.

Finish.

Section 48. A granolithic finish shall be applied to the fresh concrete as soon as the condition of its surface will permit. This finish shall consist of a mixture of one part of cement to two parts of granite, or other hard rock chips, graded from 1/8 in. to 1/2 in. in size, and shall be laid 1 1/2 in. thick. The upper surface shall be formed by means of screeds and shall be floated and troweled to a smooth surface. As soon as this surface is dry enough to receive it, a dry mixture of two parts of cement and one part of sand, free from crusher dust and particles larger than 1/8 in. shall be sprinkled over it and then the surface shall be floated and troweled. This treatment shall be repeated at least once, and where the proportion of very fine material in the aggregate necessitates it, a total of three dryer coats shall be applied. Where the placing of the dryer coat must be deferred until the day following the pouring of the concrete invert, the concrete shall be first moistened and covered with neat cement, which shall be thoroughly broomed into the concrete in the form of a thick paste.

Lining. (Alternate to Sec. 48.)

*Section 49.** Where required, the inverts shall be lined with concrete block, tile, or vitrified brick, as shown on the plans.

Brick or Concrete Block Lining.

49.1. The concrete bottom shall be accurately shaped up to a line one-half inch below the bottom of the lining and allowed to set before the lining is laid. A mixture made of one part of portland cement and three parts of sand, without the addition of water, shall be spread on the finished surface to the depth required to bring the block or brick to the required grade. The lining units shall be laid in straight lines and in a workmanlike manner and so that all joints shall be broken. After being laid, it shall be rolled with a hand roller weighing from 300 to 500 lb. or tamped until every unit shall have a solid bearing and the top of the finished work shall present a smooth and even surface and conform accurately with the shape of the invert as shown on the plans. The joints between the units shall be grouted with mortar made of one part portland cement and two parts sand and the surface shall be brushed until every joint is completely filled.

* This construction is particularly applicable to sewers having comparatively flat inverts, and is more difficult to carry out with circular sewers.

49.2. Where vitrified tile is used, the units shall be carefully bedded in wet mortar, the mortar bed to be approximately ½ in. in thickness. **Tile Lining.**

49.3. The bottom must be kept free from water until the work is completed and no water will be allowed to run over the completed work until it shall have set. **Protection.**

Section 50. The unfinished surface of the invert on which the concrete of sidewalls or arches is to be placed, shall be made as rough as possible. In unreinforced work, dovetails shall be formed as provided in Section 33. In reinforced work, where projecting bars may interfere with the formation of dovetail joints, the invert concrete shall be thoroughly cleaned by a pressure stream of water or scrubbed and every precaution shall be used to prevent earth or material from the forms falling on the surface after cleaning.

Section 51. Precaution shall be taken to prevent concrete from drying until there is no danger of cracking from lack of moisture. Concrete shall be kept moist for at least one week, unless sooner covered with earth. This may be done by a covering of wet sand, burlap, continuous sprinkling or by some other method approved by the engineer. **Keeping Concrete Moist.**

Section 52. Forms for slabs or very flat arches as in box sections or roofs of special chambers, shall remain in place for at least seven days. No load shall be placed on the concrete for fourteen days, and then only with the permission of the engineer.

Arch forms shall not be slackened until the backfilling has been carried to a height of at least one foot above the top of the arch and tamped. Arch forms shall remain in place for forty-eight hours when conditions are most favorable for the hardening of the concrete and for a longer time, as the engineer may direct during inclement weather, or where unusual conditions exist. Permission for dropping center must be secured from the engineer for each arch unit.* **Removal of Arch Forms.**

Section 53. Backfill, over and around arch sewers, shall be placed as soon as possible after the cement has set. The filling up to a plane 2 ft. above the top of the arch shall be made from the best earth and shall not contain a sufficient amount of large stones as to allow the pieces of stone to become wedged. It should be filled in layers of not over 6 in. and carefully tamped. If the remainder of the backfill is dumped from buckets, the contents of the buckets should not be allowed to fall more than 5 ft. unless the impact is broken by timber grillage. Bracing should generally be removed only when the trench below it has become completely filled and every precaution shall be taken to prevent any large slips of earth from the side of the trench onto or against the green arch. All voids left by withdrawal of sheeting shall be immediately filled with sand, by ramming with tools especially adapted to that purpose, by watering or otherwise as may be directed.

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* For small arches, 6 ft. or less, and under the most favorable conditions, forms may be dropped in 24 hours.

Section 54. During the construction of the sewer, care should be taken that no loose mortar or concrete shall be allowed to remain on the interior surface of the invert. At the completion of the work all débris shall be removed and the invert shall be left clean and smooth.

PART III-A.—MANUFACTURE OF REINFORCED-CONCRETE PIPE.

Shape. *Section 55.* Reinforced-concrete sewer pipe shall be made circular or egg shape in cross-section and circular pipe made in sizes from 24 in. to 96 in. inside diameter. Opposite diameters shall be true with a permissible variation of not more than three-quarters of one per cent.

Length. *Section 56.* Reinforced-concrete sewer pipe shall be in sections of not less than 3 ft. in length and ends so formed that when laid together and cemented they shall make a continuous and uniform line of pipe.

Material and Manufacture. *Section 57.* The provisions of Sections 1 to 12, inclusive, specifying materials for monolithic sewers, shall apply to materials for concrete pipe, and the provisions of Sections 21, 22, 23 and 27 shall apply to its manufacture. All concrete shall be of Class "A."

Forms. *Section 58.* All pipe shall be made in forms composed of sheet steel cores and casings, and cast-iron bottom and top rings which form the joint. The forms shall be rigidly held together and the core and casing so placed as to insure uniform wall thicknesses.

Curing. The top rings, cores and casings shall not be removed from the pipe until the concrete has obtained its final set.

Pipe shall not be lifted from the bottom rings until the concrete is from 60 hours to 72 hours old.

After the cores and casings have been removed from the pipe they shall be kept constantly and thoroughly wetted by sprinkling with water at least three times a day until they are removed from the bases and yarded. After being placed on the yard the pipes shall be sprinkled thoroughly at least three times a day until they are six days old.

All pipes shall be marked with the date of their manufacture and no pipe that is not 14 days old will be permitted to be laid unless it has been steam cured.

Steam Curing. Pipe may be cured by the use of wet steam in the following manner: After the pipes have been cast they shall be covered with canvas or other material known as steaming jackets and wet steam be turned into these jackets for one day after casting. Then the casings and cores may be removed and the steam again applied in the same manner for one day. After this has been done the pipes may be removed from the bases and yarded, no other curing being necessary. Steam cured pipes may be laid when they are six days old.

Samples for Testing. *Section 59.* Any or all of the following tests may be applied to samples selected by the engineer from the pipe delivered on the work. For the purposes of making such tests, the contractor shall furnish and deliver, when directed, and at the place required, five lengths of each size pipe used in the work.

Section 60. (*a*) When supported at the bottom upon a knife edge one inch in width, in such a manner that an even bearing is provided throughout the whole length, exclusive of the bell, and load is applied at the crown uniformly through a similar knife edge, the various sizes of pipe shall withstand, without signs of distress, the loads shown in Table I. **Crushing Tests.**

TABLE I.
(Knife-edge bearing.)

Diameter, in.	Load, lb. per lin. ft.
24	2149
27	2369
30	2583
33	2830
36	3080
39	3300
42	3521

(*b*) Sand bearing loads shown in Table II are equivalent in value to the knife-edge loads in Table I.

TABLE II.
(Sand bearing.)

Diameter, in.	Load, lb. per lin. ft.
24	3070
27	3370
30	3690
33	4040
36	4400
39	4710
42	5030

(*c*) When supported upon a sand saddle which extends the full length of the pipe, exclusive of the bell, and whose upper surface fits accurately the outer curved surface of the pipe, and whose width is equal to an arc of fifteen degrees, in such a manner that an even bearing is provided throughout the whole length, and the load is applied at the crown uniformly through a knife edge one inch in width, the various sizes of pipes with diameters greater than 42 in. shall withstand the following loads:

TABLE III.

Diameter, in.	Load, lb. per lin. ft.
48	3800
54	4400
60	5000
66	5500
72	6000
78	6500
84	7000
90	7500
96	8000

Absorption Test.

Section 61. The specimens for absorption tests shall be sound pieces with all edges broken, and may be from pipes broken in the crushing test. One specimen shall be selected from each pipe broken in the crushing test. They shall be from 12 to 20 sq. in. in area, and shall be as nearly square as they can be readily prepared. They shall be free from observable cracks, fissures, laminations or shattered edges.

Drying.

Preparatory to the absorption test, the specimen shall be first weighed and then dried in a drier or oven at a temperature of not less than 110 degrees C. (230 degrees F.) for not less than three hours. After removal from the drier the specimen shall be allowed to cool in dry air to room temperature and then weighed.

If the specimen is comparatively dry when taken, and the second weight closely agrees with the first, it shall be considered dry. If the specimen is wet when taken it shall be placed in the drier for a drying treatment of two hours and reweighed. If the third weight checks the second the specimen shall be redried for two-hour periods, until check weights are obtained.

Weighing.

The balance used shall be sensitive to five-tenths (0.5) grams when loaded with 1 kg., and weighings shall be read to the nearest gram. When other than metric weights are used, the same degree of accuracy shall be obtained.

Immersion.

The specimen, after final drying, cooling and weighing, shall be placed with other similar specimens in a suitable wire receptacle, packed tightly enough to prevent jostling, covered with distilled water or rain water, raised to the boiling point and boiled for five hours, and then cooled in water to room temperature.

Reweighing.

The specimen shall be allowed to drain for one minute and, the superficial moisture having been removed by towel, or blotting paper, the specimen is then placed upon the balance.

Calculation of Absorption.

The test result shall be calculated as percentage of the initial dry weight.

Reporting Results.

The results shall be reported separately for each individual specimen, together with the mean for all the specimens from the same shipment of pipe.

Identification.

Each specimen shall be marked so that it may be identified with the pipe used in the crushing test from which the specimen was taken. The marking shall be applied so that the pigment shall not cover more than one per cent of the total superficial area of the specimen.

Allowable Absorption.

The maximum allowable absorption shall be 12 per cent.

PART III-B.—LAYING THE PIPE.

Section 62. After the trench has been properly prepared the pipes shall be laid true to line and grade with the spigot end extending toward the outfall. Where pipe are laid in rock or hard ground they should be thoroughly bedded in sand or concrete. The joints shall then be filled with a mortar consisting of one part of portland cement and two parts of sand

of the quality previously specified. After the joints have been made the trench shall be carefully backfilled in thin layers and the material shall be thoroughly compacted under the haunches, around the sides and over the top of the pipes to 1 ft. above the top before the other backfilling material is thrown or dumped into the trench. This additional material must be thoroughly tamped or compacted to the top of the trench.

If wet ground or wet sand is encountered a timber platform of sufficient width and strength shall be constructed. Then the pipe shall be placed on this platform and wedged accurately to line and grade, using 2 x 6-in. wedges 18 in. long, four to each pipe. After this has been done the trench shall then be backfilled in the manner before specified.

General Note.

No attempt has been made to include herein detailed specifications, for much of the work entering into sewer construction, such as earth and rock excavation, sheeting and bracing, etc.

No specification has been included in regard to the measurement of materials or the amount of work covered in any particular compensation, other than for reinforcing steel, as it has been considered best to leave these clauses to be worked under the conditions prevailing on the particular piece of work.

RECOMMENDED RULES FOR CONCRETE SEWER DESIGN.

(To Accompany the Specifications for Concrete Sewers.)

Plain Concrete Monolithic Sewers

1. Concrete sewers without reinforcement are approved for sizes between 30 and 60 in. mean diameter. Plain concrete sewers between these sizes are to be used only in rock or hard soils. It is recommended that the minimum thickness for a diameter of 36 in. or under should be 5 in. and for a 5 ft. diameter 7 in. with intermediate sizes in proportion. These thicknesses are to be taken as a minimum for circular sewers and used only under favorable conditions.

Concrete Pipe Sewers.

2. (*a*) *Steel Reinforcement.*—The reinforcement for circular pipe shall consist of one or two rings of steel wire fabric or rods of the areas shown in Table IV. Special methods of reinforcement will be permitted if the requirements of Article 7 as to crushing strength are fulfilled.

(*b*) *Thickness.*—The thickness of pipe walls shall be as set forth in the following table, but thinner walls may be used provided the amount of steel reinforcement is increased to give sufficient strength to withstand the crushing loads specified in Article 7 of these specifications.

Reinforced Concrete Monolithic Sewers.

3. All sewers near the surface and subject to moving loads or vibration, should be reinforced. For sewers of 6 ft. or less in diameter, it is recommended that the reinforcement be ½ of 1 per cent placed near the inside at the crown, and near the outside at the springing lines.

If it appears at all possible that the horizontal pressures on the sewer might be large, reinforce for reverse stresses.

4. It is recommended that for all sewers greater than 6 ft. in diameter, several possible types of loading be assumed and stresses be calculated on

the elastic arch theory. (The methods are indicated in Turneaure & Maurer's "Principles of Reinforced Concrete," or in Metcalf & Eddy's "American Sewerage Practice," Volume I.)

5. It is also suggested that in sewers of greater than 6 ft. diameter, it may be found economical to adopt a section having a comparatively flat bottom, and an arch with or without intermediate side walls.

6. The minimum thickness of concrete in sections of this type should be 8 in. This is recommended as a factor of safety against poor placing and also to secure waterproof structures.

7. The specifications submitted provide for three classes of concrete. It is recommended that all arches be built of Class "A" concrete and that the inverts be of Class "A" concrete except in rock or very hard soils, where Class "B" concrete may be used.

TABLE IV.

Size, in.	Minimum Thickness of Shell, in.	Cross Sectional Area of Steel, per lin. ft. of Shell.
24	3	1 concentric ring of .058
27	3	" " " " .068
30	3½	" " " " .080
33	4	" " " " .107
36	4	" " " " .146
39	4	" " " " .146
42	4½	" " " " .153
48	5	2 " rings " .107
54	5½	" " " " .126
60	6	" " " " .146
66	6½	" " " " .168
72	7	" " " " .180
84	8	" " " " .208
96	9	" " " " .245

8. For reinforced work in bad ground, the designer should provide for a raft of Class "C" concrete of from 4 to 6 in. in depth, which is to be allowed to set before the reinforced structure is started. This is advisable to facilitate good workmanship and particularly to prevent contamination of the concrete around the reinforcement by mud or sand.

9. The distance from the face of reinforcing steel to the face of the concrete in monolithic sewers should be not less than 2 in.

10. In determining dimensions of concrete and reinforcement, the following working stresses should be the maximum used:

(*a*) The maximum working stress in the steel where structural grade is used should be not more than 12,000 lb. and for intermediate or hard grades, or for cold twisted bars, 16,000 lb. per sq. in.

The maximum working stress in rail steel should not exceed 16,000 lb. per sq. in.

(*b*) The maximum working stresses in concrete are based on the Report of the Joint Committee on Concrete and Reinforced Concrete and are about 25 per cent less less than the stresses there recommended.

WORKING STRESSES IN POUNDS PER SQUARE INCH.

Aggregate.	Class A.	Class B.	Class C.
Granite or trap rock	550	450	350
Gravel or hard limestone	500	400	325
Soft limestone or sandstone (if permitted)	375	300	250

Class "B" concrete is not recommended for use in the sewer proper. Soft limestone and sandstones are prohibited if the accompanying specification is rigidly carried out.

These stresses should be further reduced where construction conditions are likely to be very unfavorable to good workmanship, as in very wet or very deep trenches.

11. In all important work, specify that the reinforcement shall be held in place with steel chairs or holders and wire ties.

12. Attention is called to the fact that with sewers having comparatively flat inverts, careful consideration must be given to the distribution of load across the invert. Where soils are likely to be compressible, the weight should be taken as uniformly distributed. The stresses in such inverts should be carefully analyzed, as they are generally more severe than in the other parts of the sewer.

It will generally be advisable to provide alternate details of the invert for use in rock cuts, when resting on rock or nearly incompressible soils and for soft or wet ground.

13. Accompanying specifications for monolithic work provide for either a granolithic finish on the invert or for a lining of concrete block, brick or tile. The use of the separate lining should be considered as an additional factor of safety where unsatisfactory construction conditions are likely to affect adversely the quality of workmanship and the strength or density of the finished invert concrete.

DISCUSSION.

In the absence of the chairman, the report of the committee was presented by Mr. Coleman Meriwether, of the committee.

The President, Prof. W. K. Hatt, in the chair.

The Chairman. THE CHAIRMAN.—This report was before the convention last year and now comes in its modified form. It represents very careful work on the part of the committee, and is now before the convention for discussion. Are there any remarks?

Mr. Talbot. MR. K. H. TALBOT.—I would like to ask the committee about the speed of mixers. I find that the standard practice for a machine holding half a yard, or practically 14 cu. ft., of concrete is 16 revolutions, and that none of the machines of that size go as low as 8 revolutions. For instance, a machine that holds a full yard goes at a rate of 14 revolutions. I wondered why the minimum given in the report is 8, and why not raise that maximum to possibly 16—particularly as in the manufacture of concrete pipe you desire a consistency of concrete fairly dry, so that the question of discharge is important.

Mr. Meriwether. MR. COLEMAN MERIWETHER.—I agree with Mr. Talbot on that.

The Chairman. THE CHAIRMAN.—A motion is in order to amend the specification. Have you a motion?

Mr. Talbot. MR. TALBOT.—Mr. Chairman, I move that the second sentence of Section 23 of the specification be amended to read, "Where the character of the work will permit, concrete shall be mixed in batches of one-half to one cubic yard, and the mixed speed shall not be less than 12 nor more than 19 revolutions per minute. Where small mixers are used, the speed shall not exceed 22 revolutions per minute."

Mr. Meriwether. MR. MERIWETHER.—I second that motion.

The Chairman. THE CHAIRMAN.—It is ready for vote. Those in favor signify by saying "Aye." (Unanimous response.) Opposed. (No response.) The amendment is carried.

Mr. Lowell. MR. JOHN W. LOWELL.—In Section 5 the third sentence reads, "Crusher dust in sufficient quantity to weaken the concrete will not be permitted." In an exact specification of recommended practice, I would say that that is a very general statement, and the committee might signify definitely about what percentage of dust will be permitted. I am not in position to make a definite recommendation of what that should be, but I imagine the committee has had considerable experience and are ready to state something definite.

Mr. Chapman. MR. C. M. CHAPMAN.—It would be extremely difficult to limit the amount of crusher dust which is liable to weaken the concrete, because it depends so largely on the material used. I know of cases where 20 per cent of dust under 100-mesh produced a very strong concrete, a very excellent

concrete. Now 20 per cent in other cases would have so weakened the concrete as to be almost useless, so that it is difficult to specify a limit that would be safe in all cases without being too low for other cases. **Mr. Chapman.**

MR. LOWELL.—In Section 6 is the sentence, "Where crushed stone is used it shall have a coefficient of hardness of not less than 16." I think that is rather high. I know it would prohibit the use of limestone in a number of communities—Chicago, for instance, where only limestone is practically available. I should think that a stone having a French coefficient of 8 to 10 would be plenty hard enough for concrete sewers, and I would like to have this section amended. I make a motion to have it amended to change 16 to 8. **Mr. Lowell.**

MR. CHAPMAN.—I second that motion. **Mr. Chapman.**

MR. MERIWETHER.—On the first page of the report, the committee clearly sets out its view on that question. The committee states that placing concrete in a sewer trench is considerably more difficult than placing it in a building or in a road or highway pavement, and the sewer has different conditions of the material running through it than the highway has of the material that runs over it. I might add that the recommendations are based on Bulletins 347 and 370 of the United States Department of Agriculture. **Mr. Meriwether.**

MR. CHAPMAN.—I used to be quite opposed to the use of limestone as concrete aggregate on theoretical grounds, assuming that limestone, as it weathered out more rapidly than other stones, would disintegrate more rapidly in concrete; that being a carbonate it would be affected by heat and was not suitable for chimneys; being soft and comparatively friable, and having a low coefficient of hardness or toughness, would not be as durable as trap rock or quartz; and that it was not a suitable material for fireproofing. In all of those cases it has been proved the opposite to what I supposed would be proved before the tests were made. I do not know whether limestone will disintegrate in sewers, due to the warm, moist conditions, or not, but it is the best material I know of for chimneys. It is the best fireproofing material, or, at least, it stands very high as a fireproofing material. It makes very strong, hard concrete and of high strength—surprisingly so. It may not be suitable for sewers, but it has a pretty good record otherwise. **Mr. Chapman.**

MR. H. J. LOVE.—In view of the fact that there are a number of miles of slag concrete pipe in use and that the commercial field seems to be broadening along that line, I move that Section 5 be changed to include the words "blast-furnace slag." **Mr. Love.**

A MEMBER.—I second the motion. **A Member.**

MR. MERIWETHER.—I have had considerable experience in concrete pipe since 1906, and I agree that you can make good, hard concrete out of slag; but in our experience we found that it took a great deal more cement to make the same dense concrete out of slag than it did out of gravel, trap rock, granite or hard limestone. The slag is apparently porous and a lot of water from the concrete goes into the porous slag to make a dense con- **Mr. Meriwether.**

Mr. Meriwether. crete. We found, therefore, that we had to put as much as one bag of cement extra in a yard mix to make the same amount of concrete. For that reason I am opposed to the use of slag in reinforced-concrete pipe.

The Chairman. THE CHAIRMAN.—Those in favor of adding the words "blast furnace slag," as indicated, will say "Aye." (Response.) "No." (Responses.) The Chair is in doubt. All in favor raise their hands. (Majority responded.) Opposed. (Minority responded.) The motion prevails, and the words, "blast furnace slag" will be added to the specification. Any further discussion?

Mr. Lindau. MR. A. E. LINDAU.—I would like to ask in reference to Section 10 on reinforcement, whether the statement at the beginning of that paragraph is intended to mean that all wire fabric reinforcement shall consist of cold-drawn steel.

Mr. Meriwether. MR. MERIWETHER.—All wire is made of cold-drawn steel, and where you use wire fabric you cannot help but get cold-drawn steel.

Mr. Lindau. MR. LINDAU.—Not always. I know that it is possible to get a reinforcement that is not a cold-drawn steel wire, in the strict sense of the term, that could be used just as satisfactorily; a reinforcement that could be fabricated into a spiral, or bent or fabricated so that it could be used for pipe reinforcement and that would have an elastic limit that would conform to 55,000 lb. as a minimum. It seems to me that there is no reason for putting on a limitation there unless it is definitely a necessary limitation. I want to bring out the reason for the committee calling for a cold-drawn steel. If it is merely a matter of fabrication, or some other reason, I would like to know what the reason is. It does not appear on the surface of it why a limitation should be there.

Mr. Meriwether. MR. MERIWETHER.—I do not know of any hot-drawn steel wire that has a tensile strength of 55,000 lb. per sq. in., and I am not in a position to recommend its adoption at this time. The recommended thicknesses in these specifications in the areas of steel are based on cold-drawn steel wire with an elastic limit of not less than 55,000 lb.

Mr. Chapman. MR. CHAPMAN.—I notice a number of paragraphs here that are in the nature of a form of contract rather than a specification—"the contractor shall submit designs"—"the contractor shall provide suitable collapsible centers"—"the contractor shall do this and that"—not a specification of the work or the product, but clauses that would enter into a form of contract. It seems rather odd to me to see these things in a specification of a society. An architect will do that sort of thing, but it does not seem as though the Institute ought to do it.

Mr. Meriwether. MR. MERIWETHER.—In my experience in sewer work the specifications have always been a part of the contract.

The Chairman. THE CHAIRMAN.—The Chair understands that these specifications are recommended practice as well as definite specifications.

Mr. Lowell. MR. JOHN W. LOWELL.—In Section 7, next to the last line in the first paragraph—"All samples shall be plainly and neatly labeled." I think without any motion at all we might scratch out "and neatly." I think

that is superfluous; and in Section 10.1, under Concrete Reinforcement Bars—"(A) Billet Steel or (B) Rail Steel, except that rail steel bars may be used in sizes of 1 in. and under only, and hot-twisted bars will not be permitted." Is there any particular reason why hot-twisted bars are not permitted? **Mr. Lowell.**

MR. MERIWETHER.—Yes, in twisted bars they want cold-twisted bars. **Mr. Meriwether.**

MR. LOWELL.—Suppose the hot-twisted bars have an elastic limit of 55,000 lb. Does that not make them acceptable? **Mr. Lowell.**

MR. MERIWETHER.—It was the universal opinion of all the members of the committee that hot-twisted bars were not proper reinforcement for sewers. **Mr. Meriwether.**

MR. L. C. WASON.—May I ask if you know any reason why a hot-twisted bar that has an elastic limit of 55,000 lb., why it would not be acceptable? **Mr. Wason.**

MR. MERIWETHER.—I do not. It is merely the opinion of the committee that it should not be worded any differently. I know that in a cold-twisted bar that the best evidence that that bar is uniform steel throughout is the twisting of it cold, because if there was a soft spot in that bar in the full length, the twisting would all appear in that one spot, and I am strongly in favor of cold-twisted bars and not hot-twisted. **Mr. Meriwether.**

MR. LOWELL.—In Section 46 is the sentence, "Underdrains of agricultural tile of vitrified pipe, laid in gravel or crushed stone, shall be constructed of the size," etc. I move that we amend Section 46 so that it will read "Underdrains of agricultural tile laid in gravel or crushed stone or other suitable material," etc., omitting the words "of vitrified pipe?" **Mr. Lowell.**

A MEMBER.—I second that motion. **A Member.**

THE CHAIRMAN.—It is moved and seconded that on page 9, the words "of vitrified pipe" be omitted. Those in favor say "Aye." (Responses.) No. (No response.) So ordered. **The Chairman.**

MR. WASON.—In looking over this specification I see various clauses which seem to me to be a mixture of specification and contract. In the early specifications of the Institute, where this occurred, we were quite careful to eliminate them and to make it a specification pure and simple. I move, therefore, that the committee take this document and separate from it all those clauses which properly belong in a contract and do not come within the ordinary specification. **Mr. Wason.**

A MEMBER.—I second the motion. **A Member.**

THE CHAIRMAN.—It is moved and seconded then that this committee be instructed to separate from the document all those clauses which belong to contract and not to specification. Those in favor say "Aye." (Responses.) No. (No response.) The motion is carried. **The Chairman.**

MR. CHAPMAN.—I move that this report be referred back to the committee for discussion along the lines of the discussion this morning. **Mr. Chapman.**

THE CHAIRMAN.—The motion now is that this report be referred back to the committee for further consideration and be submitted at the next convention. Those in favor of the motion say "Aye." (Responses.) **The Chairman.**

The Chairman. Against it. (No response.) The motion is carried, then, that this report be referred back to the committee for amendment and will be brought up again before the next convention.

Mr. Mackenzie. C. L. MACKENZIE (*by letter*).—The introduction to the report states that "in introducing a coefficient of hardness in the specifications for fine and coarse aggregate . . . it was the intention . . . to prohibit entirely the use of" certain limestones.

It is not the purpose of this memorandum to discuss the correctness or propriety of ruling out this limestone, but to protest against a method which in accomplishing this specific purpose and which, also, strikes at other aggregates which may be entirely suitable for the purpose and yet be excluded from approval (and use) by an arbitrary rule.

No reference is made in the report to prepared blast-furnace slag as an aggregate, and it would seem apparent that this material has not been considered or investigated by the committee—although blast-furnace slag was very "favorably mentioned" by Sanford Thompson in his paper before the American Concrete Institute in February, 1917, in which he gave the results of his exhaustive investigation and tests. A. T. Goldbeck, also, in paper presented before the American Concrete Institute a year or two earlier, reports very favorable results from his tests in Philadelphia on blast-furnace slag as an aggregate in concrete sewers (compared with other aggregates), commenting not only on the strength but also on low absorption, or permeability, of the slag concrete.

Prof. D. A. Abrams, in Bulletin No. 1 of Lewis Institute, Chicago, on page 1, includes this statement: "We have found that the maximum strength of concrete does not depend on an aggregate of maximum density."

In addition to the investigations and tests of Mr. Thompson and Mr. Goldbeck, referred to herein, there have now been so many comparative tests made and so many million cubic yards used for all classes of concrete that the strength of concrete in which blast-furnace slag is an aggregate may fairly be said to be demonstrated.

One very exhaustive series of tests, shown in the table, page 341, is being carried on by the Pittsburgh Testing Laboratory. And there have been many other comparative tests, results of which are readily accessible, proving the high strength value of slag concrete as compared with concrete made with any other mineral aggregate now customarily used, whether gravel, limestone, trap rock, or other. And these results are obtained from blast-furnace slag which will not pass a hardness test of 16 and usually not approach it very closely.

Mr. Horner. MR. W. W. HORNER (*by letter*).—The chairman of the Committee on Concrete Sewers is in receipt of a stenographic report of the discussion on the proposed specifications for concrete sewers. Copies of this discussion have been sent to the committee members for comment, and a reply has been received from three of them. The committee cannot, therefore, present

TABLE SHOWING RESULTS OF PHYSICAL TESTS OF SLAG, STONE AND GRAVEL USED IN CONCRETE. AGE OF TESTS TWO YEARS.

P. T. L. No.	Material Used.	Mark on Cylinder.	Weight of Cylinder. lbs.	Weight of Cylinder. oz.	Crushing Strength, lbs. per sq. in.	Average.
87410	Slag Cleveland Macadam Co., Central Furnace, Cleveland, Ohio	6	62	13	4374	4357
		22	62	13	4244	
		33	63	10	4454	
87420	Slag Duquesne Slag Products Co., Duquesne, Pa.	11	65	0	4800	4882
		18	65	2	5210	
		24	64	12	4638	
87430	Slag Carnegie Steel Co., Youngstown, Ohio	19	67	0	4898	4678
		24	65	2	4212	
		8	65	4	4926	
87440	Slag Northwestern Iron Co., Mayville, Wis.	9	62	9	5040	4894
		16	62	10	4858	
		11	62	12	4786	
87450	Slag Standard Slag Co., Sharpsville, Pa.	22	64	14	5110	4740
		25	65	12	4672	
		26	65	2	4440	
87460	Gravel Allegheny River, Pittsburgh, Pa.	8	69	6	4540	4264
		16	68	12	4330	
		10	68	5	3924	
87470	Slag Cleveland Macadam Co., Cleveland Furnace, Cleveland, Ohio	9	62	0	4976	4974
		20	61	15	4896	
		26	62	12	5052	
87480	Slag Birmingham Slag Co., Ensley, Ala.	18	67	0	4928	4810
		9	66	0	5022	
		24	66	6	4480	
87490	Trap Rock Birdsboro, Pa.	17	72	7	4320	4448
		28	73	14	4330	
		16	72	15	4694	
87500	Gravel Akron, Ohio	26	68	1	3526	3576
		24	67	13	3690	
		20	67	9	3512	
87510	Crushed Granite Stock Bridge, Ga.	26	69	8	4420	4482
		9	68	8	4114	
		7	68	10	4912	
87520	Slag Duquesne Slag Products Co.,	16	63	14	4880	4718
		17	63	11	4668	
		23	63	6	4608	
87530	Slag France Slag Co., E. Toledo, Ohio	23	65	12	4932	4912
		8	67	4	4724	
		25	66	0	5080	
87540	Limestone Gates City, Ala.	46	68	5	3616	3932
		43	68	12	4020	
		42	69	14	4160	
87550	Dolomitic Limestone Kelly's Island, Cleveland, Ohio	18	70	6	5176	5260
		20	69	5	5176	
		7	69	10	5428	

Mr. Horner. a complete reply to the discussion at this time, but the following comments are offered for consideration:

In regard to the speed of concrete mixers, many members of the committee feel the manufacturers' standards are too high, and that all sizes should be run at a lower speed than at present. I appreciate that, to a large extent, we must use the machines now on the market, but to draw a specification around a manufactured standard and to continue that practice indefinitely would hardly tend to advance the best interests of the profession. The committee is willing to leave the matter of mixer's speed in the condition to which the convention amended it, and to have our original specification stand as a protest against present standards.

The discussion on the floor in regard to the recommended coefficient of hardness of not less than 16 for concrete aggregate is entirely aside from the point, and is based on the error made by Mr. Lowell that the coefficient of hardness is the same as the French coefficient of wear. It may be interesting to note that for a great many limestones in Illinois and Missouri a coefficient of hardness of 16 is likely to be accompanied by a coefficient of wear of from 7 to 10. There is, however, no general relation between the two values. The committee will attempt to present more definite data at the next meeting.

The committee is not at all certain as to how to interpret instructions use of blast-furnace slag, and prefers to let this stand as an amendment from the floor.

The committee is not at all certain as to how to interpret instructions from the floor in the manner of eliminating all "contract clauses." Apparently some members of the Institute feel that any clause containing the word "contractor" is a contract clause; for instance, Section 13, "The contractor shall furnish and place circular cast-iron frames and covers for manholes and catch-basins," etc., and might be easily read, "Manholes and catch-basins shall be provided with circular frames and covers of cast iron." Such changes as above mentioned can be very simply made, if satisfactory to the Institute. Certain other clauses, such as Section 12, pertaining to the measurement and payment of reinforcing steel, are technically contract items, but it seemed to the committee to be quite essential to the rounding out of the specifications. The committee will make some further study of the request in this matter before the next meeting.

REPORT OF COMMITTEE ON TREATMENT OF CONCRETE SURFACES.

Your committee has for some time been considering the advisability of preparing a Standard Recommended Practice for Portland Cement Stucco. Attention has been given to this matter during the past year and the committee has finally come to the conclusion that the standard specification* should be definitely supplanted by a standard recommended practice. The reasons for this change are as follows:

(1) The larger part of the present standard specification is a specification for practice. As such it contains less information than it might or should properly contain for the guidance of architects and others in preparing specifications to cover particular classes of work.

(2) Any one who uses the present standard specification must select from it paragraphs applicable to any particular job. He can make an equally good and a more intelligent selection from a standard recommended practice.

(3) Recommended practice is more flexible, and is better adapted for the presentation of new and useful information which is not properly included in a specification.

Stucco is a live subject. New developments are pending, and it is to be expected that each of the next few years will see change and improvement in the recommendations for practice. Your committee is therefore in favor of adopting a Standard Recommended Practice for Portland Cement Stucco, but prefers at this time to submit its proposal for discussion rather than final adoption. To pave the way for adoption next year it has prepared a statement setting forth some of the newer information in regard to stucco, which it presents herewith as Recommended Practice for Portland Cement Stucco.

J. C. Pearson, *Chairman.*

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* See *Journal*, American Concrete Institute, Oct.-Nov., 1914, p. 38; Jan. 1915, p. 83; *Proceedings*, A. C. I., Vol. XII, 1916, 473.

RECOMMENDED PRACTICE FOR PORTLAND CEMENT STUCCO

(To Supersede the Present Standard Specification.)

Design of the Stucco Structure.

One of the fundamental considerations in successful stucco work is a suitable design of the structure *for stucco*. The architect does not always realize that an exterior plaster of any kind merits whatever protection can legitimately be given it, that for the sake of appearance it needs more protection against leakage and drip than brick, stone, or even wood exteriors. Thus it must be recognized that stuccoed copings, cornices and horizontal or nearly horizontal surfaces are more exposed to deterioration than vertical surfaces, that attention to details of chimneys, down spouts, gutters, window sills, and overhead flashing will avoid much unnecessary staining and unsightly cracking. The committee therefore suggests the advisability of putting the paragraphs relating to "Structure" at the beginning of the specification, with one or more of these paragraphs relating to "Design" in so far as this has to do with protection for the stucco. These considerations suggest the following introductory paragraphs:

1. *Design.*—Whenever the design of the structure permits, an overhanging roof or similar projection is recommended to afford protection to the stucco. Stuccoed copings, cornices and other horizontal surfaces should be avoided whenever possible. All exposed stuccoed surfaces should shed water quickly, and whenever departure from the vertical is necessary, as at water tables, belt courses, and the like, the greatest possible slope should be detailed. Stucco should not be run to the ground whenever other treatment is possible. Should the design of the structure require this treatment, the backing should be of tile, brick, stone, or concrete, providing good mechanical bond for the stucco, and should be thoroughly cleaned before plastering. Unless special care is taken to thoroughly clean the base and each plaster coat from dirt and splash before the succeeding coat is applied, failure of the stucco may be expectd.

2. *Flashing.*—Suitable flashing should be provided over all door and window openings wherever projecting wood trim occurs. Wall copings, cornices, rails, chimney caps, etc., should be built of concrete, stone, terra cotta, or metal with ample overhanging drip groove or lip, and water-tight joints. If copings are set in blocks with mortar joints, continuous flashing should extend across the wall below the coping and project beyond and form an inconspicuous lip over the upper edge of the stucco. Continuous flashing with similar projecting lip should be provided under brick sills. This flashing should be so installed as to insure absolute protection against interior leakage. Cornices set with mortar joints should be provided wth flashing over the top. Sills should project well from the face of the stucco and be provided with drip grooves or flashing as described above for brick sills. Sills should also be provided with stools or jamb seats to insure wash of water over the face and not over the ends. Special attention should be given to the design of gutters and down spouts at returns of porch roofs where overflow will result in discoloration and cracking. A 2-in. strip should be provided at the intersection of walls and sloping roofs and flashing extended up and over it, the stucco being brought down only to the top of the strip.

3. *Preparation of Original Surface.*—All roof gutters should be fixed, and downspout hangers and all other fixed supports should be put in place before the plastering is done, in order to avoid breaks in the stucco.

Metal lath and wood lath should be stopped not less than 12 in. above grade to be free from ground moisture.

All trim should be placed in such manner that it will show its proper projection in relation to the finished stucco surface, particularly in overcoating.

MASONRY WALLS.

As the committee is of the opinion that walls of hollow tile, brick, concrete, concrete block, and similar materials, are superior to frame construction for the application of stucco, a change in the usual order of the paragraphs under "Structure" is recommended. It seems advisable, also, to include somewhat more detail in the paragraphs relating to masonry surfaces. The following revisions in the latter group are accordingly recommended:

4. *Tile.*—Tile for exterior walls, columns, etc., should be hard burned, with dovetail ragged scoring. Tile should be set in cement mortar composed of one part cement, not more than one-fifth part hydrated lime and three parts sand, by volume. The blocks should vary not more than $\frac{1}{2}$ in. in total thickness and should be set with exterior faces in line. Joints should not be raked, but mortar should be cut back to surface. Neither wire mesh or waterproofing of any type should be applied to tile walls before plastering. The surface of the tile should be brushed free from all dirt, dust and loose particles, and should be wetted to such a degree that water will not be rapidly absorbed from the plaster, but not to such a degree that water will remain standing on the surface when the plaster is applied.

5. *Brick.*—Surface brick should be rough, hard burned, commonly known as arch brick. Brick should be set in cement mortar with joints not less than $\frac{3}{8}$ in. thick, and the mortar should be raked out for at least $\frac{1}{2}$ in. from the face. The surface of the brick should be brushed free from all dust, dirt and loose particles, and should be wetted to such a degree that water will not be rapidly absorbed from the plaster, but not to such a degree that water will remain standing on the surface when the plaster is applied.

Old brick walls which are to be overcoated should have all loose, friable, or soft mortar removed from the joints, and all dirt and foreign matter should be removed by hacking, wire brushing, or other effective means. Surfaces that have been painted or waterproofed should be lathed with metal lath before overcoating.

6. *Concrete.*—Monolithic concrete walls should preferably be rough and of coarse texture, rather than smooth and dense, for the application of stucco. Walls of this type should be cleaned and roughened, if necessary, by hacking, wire brushing, or other effective means. The surface of the concrete should be brushed free from all dust, dirt, and loose particles, and should be wetted to such a degree that water will not be rapidly absorbed from the plaster, but not to such a degree that water will remain standing on the surface when the plaster is applied.

7. *Concrete Block.*—Concrete block for stucco walls should be rough and of coarse texture, but not weak or friable. Block should be set with cement mortar joints, which should be raked out or cut back even with surface. Before applying the stucco the surface should be brushed free from all dust, dirt, and loose particles, and should be wetted

to such a degree that water will not be rapidly absorbed from the plaster, but not to such a degree that water will remain standing on the surface when the plaster is applied.

In the foregoing paragraphs attention is called to the degree of wetting of the surface, which is important if best results are to be obtained. Too dry a surface will absorb the water from the plaster coat before the latter has had time to obtain its set, whereas a surface which is completely saturated is likely to be covered with a thin film of water, which will prevent proper bond of the plaster coat.

FRAME WALLS.

The general paragraph on framing needs no revision except in regard to spacing of studs. The specified 12-in. spacing is perhaps desirable in some cases, as, for example, when wire or other type of non-reinforced lath is to be back plastered, but in general such close spacing is not required.

Good bracing of the frame is important to secure the necessary rigidity. Bridging between the studs at least once in each story height is recommended whether the frame is to be sheathed or not. In the former case the bridging serves as a fire stop,* even if not necessary as bracing, and should be of the same size as the studs (usually 2 x 4-in.). In the back-plastered type of construction where sheathing is not used, bridging is required for stiffening the frame, and should be 1 in. less than the studs in depth. It should be placed horizontally, and 1 in. back of the face of the studs, in order that the back-plaster coat may be carried past the bridging without break at this point. Diagonal bracing at the corners of each wall is recommended, especially when sheathing is omitted. Such bracing may be of 1 x 6-in. boards, 6 or 8 ft. long, let into the studs on their inner side in order not to interfere with the back plastering or the interior plastering. The length of the corner bracing will, of course, depend to some extent on the location of window or other openings.

When sheathing is used, it should be laid horizontally and not diagonally across the studs. The stucco test panels erected at the Bureau of Standards in 1915 and 1916 have demonstrated conclusively that diagonal sheathing tends to crack the overlying stucco by setting up strains in the supporting frame. This result is undoubtedly due to the shrinkage of the sheathing, and whatever benefit might be anticipated from the more effective bracing provided by diagonal sheathing appears to be more than offset by the shrinkage effect. Diagonal sheathing is also less economical than horizontal sheathing, both in material and labor.

Waterproofing of the faces of the studs in back-plastered construction seems to be ineffective and unnecessary, and its elimination is recommended.

The proper type and depth of furring is a question on which information is desired. If metal lath is applied over sheathing and the commonly recommended practice of filling with mortar the space between lath and sheathing is to be followed, there seems to be no good reason for using furring deeper

* The committee takes this opportunity of recommending fire stopping of frame structures as described in the National Board of Fire Underwriters' book entitled "Dwelling Houses."

than $\frac{3}{4}$ in. On the other hand 1 x 2-in. wood furring is widely used for both metal and wood lath, and there are good arguments both for and against this type of furring. The question of the proper length and gage of staples for metal lath is involved with that of furring. The entire subject needs investigation. At the present time the committee is not sufficiently well informed to recommend a change in the existing paragraphs of the specification, aside from reducing the depth of furring from $\frac{1}{2}$ to $\frac{3}{8}$ in.

Metal lath should be specified by weight rather than by gage, and should be always galvanized or painted. Galvanized lath is a good investment in most cases, and is to be recommended in preference to painted lath, unless the method of applying the stucco is such as to insure complete embedment of the metal, as, for example, in the back-plastered type of construction.

The results of tests and field observations indicate that more attention should be given to the application of lath to exterior surfaces. Cracks frequently develop in stucco over laps or at junctions of metal and wire lath, indicating a weakness at these points. This may be due in part to reduced thickness of the stucco where the lath is lapped, or to insufficient tying and fastening at the joints. The ideal job of lathing would obviously be that in which the lath forms a uniform fabric over the structure, without seams or lines of weakness, and with equal reinforcing value in all directions. This ideal condition cannot be realized, but evidence is at hand to indicate that butted and laced, or well-tied horizontal joints are better than lapped joints, and in the case of ribbed lath, that carefully locked joints are better than lapped joints. Vertical joints must almost of necessity be lapped, but the joints may be made secure if they occur over supports and are well stapled at frequent intervals.

The use of wood lath as a base for cement stucco finds many advocates and many opponents, and more field and test data should be available before the evidence for and against wood lath can be carefully weighed. Further information is desired in regard to the type of wood lath best suited for cement stucco. In some of the most satisfactory work reported by the committee, the lath were of white pine 1 in. wide and $\frac{1}{2}$ in. thick. Both material and size were here unusual, but the committee is of the opinion that this type of narrow lath is worthy of consideration. For want of information as to the practicability of specifying any particular kind of wood and unusual dimensions, no change is suggested at the present time. It may be stated, however, that nearly all of the test panels of wood lath erected at the Bureau of Standards developed large cracks, in such manner as to suggest that narrower lath (those used were $1\frac{3}{8}$ in. wide) with wider keys and heavier nailing would have given better results. The tests also indicate that counter lathing in which the lath are applied lattice fashion produces no more satisfactory results than plain lathing. In view of the much greater cost of counter lathing the committee recommends that reference to this type of application be omitted from specifications.

More information is needed on the subject of insulation, particularly in connection with the back-plastered type of construction. At the present time, the warmth of the back-plastered stucco house in comparison with that

of the sheathed house is questioned by some, but the available evidence seems to indicate that where insulation has been provided as specified, generally satisfactory results have been obtained. The committee might well undertake to learn what the experience of owners of back-plastered stucco houses has been, in view of the fact that this type of construction has been quite widely used in recent years.

On the basis of the foregoing remarks the following paragraphs relating to the frame structure are suggested:

8. *Framing.*—Studs spaced not to exceed 16-in. centers should be run from foundation to rafters without any intervening horizontal members. The studs should be tied together just below the floor joists with 1 x 6-in. boards which should be let into the studs on their inner side, so as to be flush and securely nailed to them. These boards will also act as sills for the floor joists, which, in addition, should be securely spiked to the side of the studs.

9. *Bracing.*—The corners of each wall should be braced diagonally with 1 x 6-in. boards let into the studs on their inner side, and securely nailed to them.

*(*a*) At least once midway in each story height the studs should be braced horizontally with 2 x 3-in. bridging set 1 in. back of the face of the studs.

(*b*) No bridging is necessary.

10. *Sheathing.*—(*a*) The lath should be fastened direct to the studding and back plastered, and no sheathing is used.

(*b*) Sheathing boards should not be less than 6 in. nor more than 8 in. wide, dressed on one or both sides to a uniform thickness of $\frac{7}{8}$ in. They should be laid horizontally across the wall studs and fastened with not less than two 8*d*. nails at each stud.

11. *Inside Waterproofing.*—(*a*) No waterproofing is necessary.

(*b*) Over the sheathing boards should be laid in horizontal layers, beginning at the bottom, a substantial paper, well impregnated with tar or asphalt. The bottom strip should lap over the baseboard at the bottom of the wall, and each strip should lap the one below at least 2 in. The paper should lap the flashings at all openings.

12. *Furring.*—Metal Lath. When furring forms an integral part of the metal lath to be used, then separate furring as described in this paragraph is omitted.

(*a*) Galvanized or painted $\frac{3}{8}$ in. crimped furring, not lighter than 22-gage or other shape giving equal results, should be fastened direct to the studding, using $1\frac{1}{4}$ in. x 14-gage staples spaced 12 in. apart.

(*b*) Galvanized or painted $\frac{3}{8}$-in. crimped furring not lighter than 22-gage or other shape giving equal results, should be fastened over the sheathing paper and directly along the line of the studs, using $1\frac{1}{4}$ in. x 14-gage staples spaced 12 in. apart. The same depth of furring should be adhered to around curved surfaces, and furring should be placed not less than $1\frac{1}{2}$ in. nor more than 4 in. on each side of and above and below all openings.

Wood Lath. Furring 1 x 2-in. should be laid vertically 12 in. on centers over the sheathing paper and nailed every 8 in. with 6*d*. nails.

13. *Lath.*—Metal lath should be galvanized or painted expanded lath weighing not less than 3.4 lb. per sq. yd.

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*Paragraphs marked (*a*) apply only to back-plastered construction in which sheathing is omitted Paragraphs marked (*b*) apply only to sheathed walls. The paragraphs relating to bridging assume that studs are 2 x 4 in. Larger sizes would require correspondingly larger bridging.

Wire lath should be galvanized or painted woven wire lath, not lighter than 19-gage, $2\frac{1}{2}$ meshes to the inch, with stiffeners at 8-in. centers.

Wood lath should be standard quality, narrow plastering lath 4 ft. long and not less than $\frac{3}{8}$ in. thick.

14. *Application of Lath.*—Metal Lath. Lath should be placed horizontally driving galvanized staples $1\frac{1}{4}$ in. x 14-gage not more than 8 in. apart over the furring or stiffeners. Vertical laps should occur at supports and should be fastened with staples not more than 4 in. apart. Horizontal joints should be locked or butted and tightly laced with 18-gage galvanized wire.

Wood Lath. Lath should be placed horizontally on the furring with $\frac{1}{2}$-in. openings between them. Joints should be broken every twelfth lath. Each lath should be nailed at each furring with 4*d.* nails.

15. *Corners.*—Metal Lath. The sheets of metal lath should be folded around the corners a distance of at least 3 in. and stapled down, as applied. The use of corner bead is not recommended.

Wood Lath. At all corners a 6-in. strip of galvanized or painted metal lath should be firmly stapled over the wood lath with $1\frac{1}{4}$ in. x 14-gage galvanized staples.

16. *Spraying.*—Before applying the first coat of plaster, wood lath should be thoroughly wetted, but water should not remain standing on the surface of the lath when the plaster is applied.

17. *Insulation.*—The air space in back-plastered walls may be divided by applying heavy building paper, quilting, felt, or other suitable insulating material between the studs, and fastening it to the studs and bridging by nailing wood strips over folded ends of the material. This insulation should be so fastened as to leave a greater air space next to the interior plaster. Care should be taken to keep the insulating material clear of the outside plaster, and to make tight joints against the wood framing at the top and bottom of the space and against the bridging.

MATERIALS.

The only change in the paragraphs under this heading which is warranted at the present time is that relating to lime. It is believed that hydrated lime should be specified to the exclusion of lump lime, chiefly for the reason that lime which is slaked on the job cannot as a rule be so thoroughly hydrated and so thoroughly mixed in the mortar as the mechanically hydrated product

The committee also calls attention to the fact that "blended cements" composed of portland cement ground and mixed with finely divided sand or other suitable materials, may properly find a place among the stucco materials of the future. One of the experimental panels erected at the Bureau of Standards in 1916 was plastered with a mixture of this type, and has a very high rating, both in appearance and freedom from defects. Further experiments along this line are planned for the future.

The paragraphs relating to materials are as follows:

18. *Cement.*—The cement should meet the requirements of the standard specifications for portland cement of the American Society for Testing Materials, and adopted by this Institute. (Standard No. 1.)

19. *Fine Aggregate.*—Fine aggregate should consist of sand, or screenings from crushed stone or crushed gravel, graded from fine to coarse, passing when dry a No. 8 screen. Fine aggregate should preferably be of silicious materials, clean, coarse and free from loam, vegetable

or other deleterious matter. In this connection reference may be made to the recently developed colorimetric test for detecting the presence of organic matter in sands.

20. *Hydrated Lime.*—Hydrated lime should meet the requirements of the standard specifications for hydrated lime of the American Society for Testing Materials.

21. *Hair or Fiber.*—There should be used only first quality long hair, free from foreign matter, or a long fiber well combed out.

22. *Coloring Matter.*—Only mineral colors should be used which are not affected by lime, portland cement, or other ingredients of the mortar, or the weather.

23. *Water.*—Water should be clean, free from oil, acid, strong alkali or vegetable matter.

PREPARATION OF MORTAR.

The importance of proper and thorough mixing of the ingredients of the mortar cannot be too strongly emphasized. Believing that machine mixing is superior to hand mixing, the committee suggests the rewording of the paragraph on mixing. The use of hair or fiber is considered optional, and when used the method of incorporation should be such as to insure good distribution and freedom from clots. The maintenance of proper and uniform consistency should be insured by measurement of the water as well as of the other ingredients of the mortar. The question of retempering mortar is one which will bear further investigation. At the present time sufficient information is not available to warrant a change in the paragraph on retempering. These paragraphs are as follows:

24. *Mixing.*—The ingredients of the mortar should be mixed until thoroughly distributed, and the mass is uniform in color and homogeneous. The quantity of water necessary for the desired consistency should be determined by trial, and thereafter measured in proper proportion.

Machine Mixing. The mortar should preferably be mixed in a suitable mortar mixing machine of the rotating drum type. The period of machine mixing should be not less than five minutes after all the ingredients are introduced into the mixer.

Hand Mixing. The mixing should be done in a water-tight mortar box, and the ingredients should be mixed dry until the mass is uniform in color and homogeneous. The proper amount of water should then be added and the mixing continued until the consistency is uniform.

25. *Measuring Proportions.*—Methods of measurement of the proportions of the various ingredients, including the water, should be used which will secure separate uniform measurements at all times. All proportions stated should be by volume. A bag of cement (94 lb. net) may be assumed to contain 1 cu. ft. 40 lb. may be assumed as the weight of 1 cu. ft. of hydrated lime. Hydrated lime should be measured dry, and should not be measured nor added to the mortar in the form of putty.

26. *Retempering.*—Mortar which has begun to stiffen or take on its initial set should not be used.

27. *Consistency.*—Only sufficient water should be used to produce a good workable consistency. The less water, the better the quality of the mortar, within working limits.

MORTAR COATS.

Practice varies widely in the mixture and application of stuccos. The use of hair, lime, and waterproofing materials, the variations in the mixtures

for the different coats, the number and thickness of the coats, the intervals between the coats, the degree of wetting of the undercoats, and the precautions necessary in protecting the coats from too rapid drying, are details subject to question, and all will stand further investigation. However, the study of the experimental panels at the Bureau of Standards has yielded considerable information on some of these points

One of the most important indications from these panels is that lean mixtures containing well graded aggregate give better results than those commonly specified. Mixtures as lean as one part of cement to six or seven parts of graded aggregate have given excellent results in these tests. The committee is of the opinion that the volume change of rich mortars is accountable for much of the unsightly cracking of stuccoes, and that no mixture should be used in which the proportion of cement is greater than one part to three parts of fine aggregate.

The effect of hydrated lime in cement stucco has also been given considerable attention, and the conclusion which is forcing itself upon the committee is that hydrated lime does not improve the structure of the stucco, but by imparting better working quality to the mortar, reduces the cost of application. On the other hand there is evidence that not more than 20 per cent of hydrated lime, by volume of the cement, should be added to cement stucco if the best results are to be obtained.

There seems to be no good reason for varying the composition of the different coats, but if a variation is to be specified, the scratch coat should logically be the strongest mixture followed by a leaner brown coat, and a still leaner finish. No greater mistake has ever been made in stucco application than the use of a strong brown coat over a weak base or a weak scratch coat. The not uncommon practice of applying a strong brown coat over a lime mortar scratch coat has been responsible for many stucco failures.

The suggestion that the finish coat should logically be leaner than the undercoats immediately brings up the waterproofing question. There are two fundamental points to be considered in this connection; first, that the lean coat is not necessarily lacking in density, and second, that the waterproofing problem in good cement stucco is not one of overcoming permeability, but rather of reducing absorption. The entire question hinges on absorption, and the evidence at hand indicates that a moderate degree of absorption is a much more preferable condition than a surface covered with craze and map cracks produced by the use of a too rich or wrongly manipulated finishing coat. Any waterproofing treatment that alters the natural texture and color of the stucco may be dismissed from consideration, and the merit of any integral waterproofing in stucco is exceedingly difficult to determine.

The question as to number and thickness of coats may be best answered by assuming that each coat of stucco has its own particular function. The scratch coat is the first applied, and its purpose is to form an intimate bond and a secure support for the body of the stucco. On metal lath it also serves as a protective coat, and it should therefore be strong and not too lean. The use of hair or fiber is of questionable value. Hair or fiber should not be used when the space back of the lath is to be filled, and is probably not a necessary

ingredient in any case. The committee at the present time would sanction its use only in scratch coats on wood lath, or on metal or wire lath that is to be back plastered, or on metal or wire lath that is applied over furring deeper than $\frac{3}{8}$ in. The thickness of the scratch coat should average about $\frac{1}{4}$ in. over the face of the lath.

The function of the second coat (commonly called the brown or straightening coat) is to establish a true and even surface upon which to apply the finish. It forms the body of the stucco, and must fill the hollows and cover the humps of the scratch coat. For this reason an average thickness of $\frac{3}{8}$ in. to $\frac{1}{2}$ in. will usually be required. The brown and finish coats, or the scratch and brown coats, are sometimes combined in two-coat work, which is permissible when the base upon which the stucco is applied is fairly true and even, or when, on account of cost considerations, the best obtainable finish is not required. It is difficult, however, to obtain a satisfactory finish on a coat which runs $\frac{1}{2}$ in. or more in thickness, since the tendency of a heavy coat to bag and slip is likely to produce an uneven surface.

The finish coat serves only a decorative purpose and has no structural value. Its function is solely to provide an attractive appearance, and any mixture or any method of application that may detract from the appearance, or in any way injure its permanency, should be avoided. Herein lies the argument for lean mixtures, which are more likely to be free from unsightly defects than rich mixtures, and are also more likely to improve in appearance under the action of the weather. The finish coat should be as thin as possible consistent with covering capacity, and may vary from $\frac{1}{8}$ in. to $\frac{3}{8}$ in. in thickness, depending upon the type employed.

It is obvious from the foregoing that first-class stucco should be three-coat work, each coat serving its own particular purpose. The bond between the brown coat and the scratch coat needs to be strong in order to carry the weight of the body of the stucco, and for this reason it is now considered preferable to apply the brown coat the day following the application of the scratch coat. Except in dry or windy weather little wetting of the scratch coat should be necessary when the brown coat is to follow within 24 hours. A slight degree of absorption of "suction" in the scratch coat, is probably better than complete saturation, for the brown coat, as well as the others, is necessarily mixed with a larger quantity of water than it requires for maximum strength. The removal of a portion of this excess water by the suction of the undercoat not only improves the quality of the coat, but also insures a better bond by tending to draw the fine particles of the cement into the pores and interstices of the undercoat.

Whereas the interval between the brown coat and scratch coat, as recommended above, is relatively short, the interval before applying the finish coat should be as long as permissible under the conditions of the work. The reason for thus delaying the application of the finish is to enable the body of the stucco to obtain its initial shrinkage and a nearer approach to its final condition of strength and hardness, before being covered with the surface coat. The bond of the latter needs to be intimate rather than of maximum strength, and if the body of the stucco has been allowed to thoroughly set

and harden, it may be assumed that the finish coat is less likely to be disturbed by subsequent volume changes in the undercoats. A week or more should elapse between the application of the brown and finish coats.

The finish coat should be applied over a damp, but not saturated, undercoat, for excess water is likely to injure the bond seriously. Certain types of finish, such as the wet mixtures used for sand spraying, or for the "spatter dash" finish, may preferably be applied to a fairly dry undercoat, since suction must be depended upon to prevent streakiness and muddy appearance. The fact that finishes of this type applied in this manner may set and dry out with little strength is not serious; they gradually attain sufficient hardness with exposure to the weather.

Curing of the undercoats by sprinkling, and protection of finish coats against sun, wind, rain and frost by means of tarpaulins are always to be recommended. This is not always feasible, however, and the architect should be content to specify and insist upon reasonable precautions. The application of cement stucco in freezing weather should be avoided, and in fact temperatures slightly above the freezing point may allow frost to form on a damp wall. The application of stucco under such conditions is likely to result in failure.

The foregoing discussion suggests the following revised paragraphs relating to mortar coats:

28. *Mortar.*—All coats should contain not less than 3 cu. ft. of fine aggregate to 1 sack of portland cement. If hydrated lime is used, it should not be in excess of one-fifth the volume of cement. Hair or fiber should be used in the scratch coat only on wood lath, or metal or wire lath which is applied over sheathing and is separated therefrom by furring deeper than $\frac{3}{8}$ in.

29. *Application.*—The plastering should be carried on continually in one general direction without allowing the plaster to dry at the edge. If it is impossible to work the full width of the wall at one time, the joining should be at some natural division of the surface, such as a window or door.

The first coat should thoroughly cover the base on which it is applied and be well troweled to insure the best obtainable bond. Before the coat has set it should be heavily cross-scratched with a saw-toothed metal paddle or other suitable device to provide a strong mechanical key.

The second coat should be applied whenever possible on the day following the application of the scratch coat. The first coat should be dampened if necessary, but not saturated, before the second coat is applied. The second coat should be brought to a true and even surface by screeding at intervals not exceeding 5 ft., and by constant use of straightening rod. When the second coat has stiffened sufficiently, it should be dry floated with a wood float and lightly and evenly cross-scratched to form a good mechanical bond for the finish coat. The day following the application of the second coat, and for not less than three days thereafter, the coat should be sprayed or wetted at frequent intervals and kept from drying out.

In back-plastered construction the backing coat should preferably be applied directly following the completion of the brown coat. The keys of the scratch coat should first be thoroughly dampened, and the backing coat then well troweled on to insure filling the spaces between the keys and thoroughly covering the back of the lath. The backing

coat should provide a total thickness of plaster back of the lath of $\frac{5}{8}$ in. or $\frac{3}{4}$ in., and should finish about $\frac{1}{4}$ in. back of the face of the studs.

The finish coat should be applied not less than a week after the application of the second coat. Methods of application will hereinafter be described under "finish."

30. *Two-Coat Work.*—Whenever two-coat work is required the first coat should preferably be "doubled," that is, as soon as the first coat is stiff enough, it should be followed by a second application of mortar, and this should then be treated as described for the second coat under paragraph 29. The finish coat should be applied not less than a week after the application of the first coat.

31. *Drying Out.*—The finish coat should not be permitted to dry out rapidly, and adequate precaution should be taken, either by sprinkling frequently after the mortar is set hard enough to permit it, or by hanging wet burlap or similar material over the surface.

32. *Freezing.*—Stucco should not be applied when the temperature is below 32° F., nor under any conditions such that ice or frost may form on the surface of the wall.

Finish.

It is practically impossible to specify in written paragraphs the methods by which successful finishes are obtained. The quality of these depends upon the knowledge and skill of the plasterer, and the specification writer must content himself with a brief description of the several types. In the finishing of stuccoes, however, there are certain causes and effects which should be more generally recognized, and the committee believes that a brief discussion of these will help to explain the limitations of the commonly used finishes and indicate the methods to be pursued in the attempt to develop better finishes.

In an earlier paragraph the defects resulting from the expansion and contraction of rich mortars has been referred to. The chance of such defects occurring must be greatest in the finish coat, which is directly exposed to the extremes of moisture and temperature variations. The hope of overcoming these defects lies mainly in the use of leaner mixtures, in which the tendency to movement is cut down as the proportion of cement is reduced. The problem therefore is to use less cement and at the same time retain the necessary density by improved gradation of the aggregate. Considerable success has already attended experiments along this line, and even better results are anticipated in the future.

All that may be accomplished in this direction, however, will hardly permit a smooth troweled finish to be used. This treatment produces a concentration of fine material at the surface, which will almost inevitably develop fine cracks. In the course of time these cracks will collect soot and dirt and become conspicuous and unsightly At best the smooth troweled finish is not to be recommended. and specifications should eliminate all reference to it.

The dash finishes—such as the sand spray. which is obtained by applying a mixture of sand, cement and water with a whisk broom or long fiber brush, or the spatter dash, which is usually a thin mortar containing coarse sand or stone screenings thrown from a paddle, or the rough-cast which is a mixture of pebbles and cement grout thrown from a paddle or the back of a trowel—

are all relatively rich in cement and all develop fine cracks to a very marked degree, but the rough texture of the surfaces masks these defects, and the type is therefore generally satisfactory and very widely used. The use of these finishes is in general to be recommended, unless the work is done by a stucco specialist whose skill and experience qualifies him to execute the more difficult finishes to be discussed in the following paragraphs.

The chief objection to the dash finishes as above described is their rather cold, unbroken cement color, which may be relieved and improved to a considerable extent by the judicious use of mineral pigments. Another means of varying the monotony of the natural grays and whites of the cement is by the use of the dry dash finishes in which clean pebbles or stone chips are thrown against the fresh mortar of the finishing coat while it is still soft. When the dry dash is well selected and the particles thickly and uniformly distributed over the surface, the finish thus obtained is pleasing and possesses decidedly more life and character than the wet dashes.

The sand-float finish deserves special consideration because it promises to be one of the most satisfactory finishes of the future. Due to the use of rich mixtures the sand float finish has usually developed defects similar to those experienced with the smooth troweled finishes, differing from the latter only in degree. Sand-floated stuccoes which have been covered with paint are to be found in every community, and this alone is sufficient evidence of unskilful manipulation of this finish and of the unsatisfactory results that have been obtained. In the experiments carried out at the Bureau of Standards, the sand-float finish was found to be most satisfactory on mixtures containing not more than 1 part of portland cement to 4 parts of fine aggregate, and mixtures as rich as 1:3, with a small addition of hydrated lime were satisfactory as a rule only when the final floating was delayed until the mortar had well stiffened. In this manner the concentration of fine material in the surface was prevented. This experience confirms the necessity for using leaner mixtures than have been specified heretofore, and for removing the cement from the surface by mechanical or other means, if the sand-float finish is to come into its own.

There is no hard and fast line between the sand-float finish and the exposed aggregate finish, since in the final water floating process of the former the aggregate is left sufficiently exposed to modify and improve the tone of the finished wall. When the sand floated surface is further improved by an acid wash, the grains of the aggregate are cleanly exposed. It seems preferable in classification, however, to limit the exposed aggregate finishes to those in which coarser aggregates are employed than would be feasible for the sand-float finish. Thus defined, the exposed aggregate finish is obtained by the application of a coarse mortar containing carefully selected and graded aggregates, so that when the latter are exposed by brushing and cleaning, the resulting texture resembles that of cast concrete which has been subjected to similar surface treatment. One of the members of your committee has recently developed a stucco finish of this type which is being applied to the Field House in East Potomac Park, Washington, D. C., over terra-cotta tile. The color and texture of this finish, produced entirely by the aggregate, is the same as that of the concrete trim of the building.

The committee believes that the exposed aggregate finish will ultimately be developed and come into general use as the most satisfactory of stucco finishes. Commonly available aggregates are capable of giving very beautiful effects, and it has been demonstrated that the colors and tones thus obtained improve with exposure to the weather. A considerable amount of experimental work remains to be done before the best methods of producing these finishes can be specified, but the committee hopes that by hearty co-operation of those agencies interested in the development of improved stuccoes, this experimental work may soon be undertaken and carried through to a successful conclusion.

No changes in the wording of the paragraphs relating to finish are recommended at the present time, which are as follows:

> 33. *Stippled.*—The finishing coat should be troweled smooth with a metal trowel with as little rubbing as possible, and then should be lightly patted with a brush of broom straw to give an even, stippled surface.
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> 34. *Sand Floated.*—The finishing coat, after being brought to a smooth, even surface, should be rubbed with a circular motion of a wood float with the addition of a little sand to slightly roughen the surface. This floating should be done when the mortar has partly hardened
>
> 35. *Sand Sprayed.*—After the finishing coat has been brought to an even surface, it should be sprayed by means of a wide, long fiber brush—a whisk broom does very well—dipped into a creamy mixture of equal parts of cement and sand, mixed fresh at least every 30 minutes, and kept well stirred. This coating should be thrown forcibly against the surface to be finished. This treatment should be applied while the finishing coat is still moist and before it has attained its early hardening, that is, within 3 to 5 hours. To obtain lighter shades add hydrated lime not to exceed 10 per cent of the weight of the cement.
>
> 36. *Rough-Cast or Spatter Dash.*—After the finishing coat has been brought to a smooth, even surface with a wooden float and before finally hardened, it should be uniformly coated with a mixture of one of sack cement to 3 cu. ft. of fine aggregate thrown forcibly against it to produce a rough surface of uniform texture when viewed from a distance of 20 ft. Special care should be taken to prevent the rapid drying out of this finish by thorough wetting down at intervals after stucco has hardened sufficiently to prevent injury.
>
> 37. *Pebble Dash.*—After the finishing coat has been brought to a smooth, even surface, and before it has begun to harden, clean round pebbles, or other material as selected, not smaller than $\frac{1}{4}$ in. or larger than $\frac{3}{4}$ in. and previously wetted, should be thrown forcibly against the wall so as to embed themselves in the fresh mortar. They should be distributed uniformly over the mortar with a clean wood trowel, but no rubbing of the surface should be done after the pebbles are embedded.
>
> 38. *Exposed Aggregates.*—The finishing coat should be composed of an approved, selected coarse sand, crushed marble, or granite or other special material, in the proportion given for finishing coats, and within 24 hours after being applied and troweled to an even surface, should be scrubbed with a stiff brush and water. In case the stucco is too hard, a solution of one part hydrochloric acid in four parts of water by volume can be used in place of water. After the aggregate particles have been uniformly exposed by scrubbing, particular care should be taken to remove all traces of the acid by thorough spraying with water from a hose.
>
> 39. *Mortar Colors.*—When it is required that any of the above finishes should be made with colored mortar not more than 10 per cent

of the weight of portland cement should be added to the mortar in the form of finely ground mineral coloring matter.

A predetermined weight of color should be added dry to each batch of dry fine aggregate before the cement is added. The color and fine aggregate should be mixed together and then the cement mixed in. The whole should be then thoroughly mixed dry by shoveling from one pile to another through a ¼-in. mesh wire screen until the entire batch is of uniform color. Water should then be added to bring the mortar to a proper plastering consistency.

In conclusion the committee desires to state its conviction that while portland cement stucco may develop certain small defects which cannot always be guarded against, the product may be depended upon, if applied in accordance with the foregoing recommended practice, to be structurally sound, durable, and capable of giving satisfactory service with little or no outlay for repairs or maintenance. The improvements which may be expected are those pertaining to appearance, and those tending to eliminate smaller faults, which, although structural in themselves, are yet more damaging to appearance than to permanency.

D. K. BOYD,
E. D. BOYER,
W. H. BOUGHTON,
C. M. CHAPMAN,
W. CLAY,
J. J. EARLEY,
J. E. FREEMAN,
J. B. ORR,
J. C. PEARSON, *Chairman.*

DISCUSSION OF REPORT OF COMMITTEE ON TREATMENT OF CONCRETE SURFACES.

(Presented by the Chairman of the Committee, President W. K. Hatt, in the Chair.)

DISCUSSION.

The Chairman. THE CHAIRMAN.—The Chair might remark that he thinks the Institute is very fortunate, indeed, in having the splendid service of this committee during the past two years.

A Member. A MEMBER.—I move that the report as submitted be printed as Recommended Practice in place of our former specification.

A Member. A MEMBER.—I second the motion.

The Chairman. THE CHAIRMAN.—It is moved and seconded that this report of the committee be received and printed as the Recommended Practice of the Institute.

Mr. Woolson. MR. I. H. WOOLSON.—Referring to the head of "Frame Walls," in the second paragraph, where reference is made to fire stopping, it says: "In the former case the bridging serves as a fire stop, even if not necessary as bracing, and should be of the same size," etc. I would like to ask if the bridging is in, if that is all the fire stopping that would be required.

Mr. Pearson. MR. J. C. PEARSON.—This matter has been discussed to some extent, and we believe that the fire stop should be at floor levels, although that does not concern this committee's work. We have only taken the opportunity to recommend that attention be given this. We have no jurisdiction as far as fire stopping is concerned.

Mr. Clay. MR. CLAY.—I move as an amendment to the resolution that the chairman of this committee be empowered to change that footnote so that it will carry the information he discussed from the floor.

A Member. A MEMBER.—I second the motion.

The Chairman. THE CHAIRMAN.—Those in favor say "Aye." (Responses.) No. (No response.) The motion is carried.

Mr. Libberton. MR. J. L. LIBBERTON.—I move that paragraphs 4, 5, 6 and 7 be placed in alphabetical order under the heading "Masonry Walls," and that the first sentence be made to read, under paragraph 4, "Tile for exterior walls, columns, etc., should be hard-burned clay, with dovetail ragged scoring or concrete, which is not weak or friable." Also that paragraph 5 should read: "Surface brick should be rough, hard-burned clay, commonly known as arch brick or of concrete not weak or friable."

A Member. A MEMBER.—I second the motion.

The Chairman. THE CHAIRMAN.—It is moved and seconded then that these paragraphs be listed in alphabetical order, and that the words "or concrete not weak or friable" be inserted in the second line of paragraph 4 and the second line of paragraph 5.

MR. PEARSON.--That is such an obvious device to bring the concrete to the fore there that I would prefer not to see it. This was put in here as a group without reference to the order in the group, and concrete is a less common thing. The other things are more in the order of their uses. I believe it should remain as it is. I don't believe there is any aspersion on concrete because it is down at the bottom. Mr. Pearson.

THE CHAIRMAN.—Will you separate your motion, Mr. Libberton, the first part in reference to listing these in alphabetical order, and the second motion, which is an amendment to the document. The Chairman.

MR. LIBBERTON.—I move that paragraphs No. 4, 5, 6 and 7 be placed in alphabetical order. Mr. Libberton.

A MEMBER.—I second the motion. A Member.

THE CHAIRMAN.—Those in favor say "Aye." (Responses.) No. (Responses.) The Chair is in doubt. All those in favor, please raise their hands. (Majority responded.) The Chairman.

MR. LIBBERTON.—I move that paragraph 4 be changed to read: "Tile for exterior walls, columns, etc., should be hard-burned clay, with dovetail ragged scoring, or of concrete not weak or friable." Under paragraph 5: "Surface brick should be rough, hard-burned clay, commonly known as arch brick or of concrete not weak or friable." Mr. Libberton.

THE CHAIRMAN.—That motion has been seconded. It is ready for a vote. Those in favor say "Aye." (Responses.) Against it. (No response.) The motion is carried. Is there any further discussion? The Chairman.

THE CHAIRMAN.—Is there any further discussion? If not, we will now take a vote on the original motion as amended—that this report be printed as Recommended Practice for Portland Cement Stucco. Those in favor say "Aye." (Response.) Against it. (No response.) It is a vote. The Chairman.

REPORT OF THE COMMITTEE ON REINFORCED-CONCRETE HIGHWAY BRIDGES AND CULVERTS.

The work of the Committee on Highway Bridges and Culverts during the past four months is given herewith as a matter of record since a late start and the impossibility of a meeting of its membership to discuss the data collected precluded definite recommendations of the full program contemplated.

Recommendations are made relating to the important assumptions necessary to fixing the effective width carrying a concentrated load on a simple reinforced-concrete slab superstructure since these are logical conclusions of a series of convincing tests carried on since 1912.

There is a wide field of endeavor for this committee, so wide, in fact, that it seems necessary to subdivide and enlarge the committee membership in the future to obtain active participation and results commensurate with its importance. Within a very short time reinforced-concrete bridge construction has had a very rapid growth, during which there has been injected much individuality which the art invites. This has resulted in seemingly wide differences in general practice pertaining to design, little less in other fundamentals of the development. Therefore, there is much to be desired in the establishment of greater uniformity in the practice of concrete bridge construction.

The State of Iowa has an admirable law, making it the duty of the State Highway Commission to devise standard plans of the small type structure and specifications governing all concrete construction adapted to the needs of the several counties in the state, requiring the county to construct all this work in accordance therewith.

Many valuable recommendations have been advanced by our former committees, and a review of these reports leads to the thought that some permanent form in submitting the report of this committee should be devised similar to the manner in which the structural steel bridge work has been treated. The work could be divided into four general topics as follows:

1. *Recommended Practice.*—A few of the subjects that could be treated under this heading, with a view to more uniform practice, would be recommendations governing (*a*) Preparation of survey and plans, (*b*) Arrangement, spacing and nomenclature of the reinforcing steel amplified with illustrations, (*c*) Best position of laps in long-span girders and arrangement of bars to provide for negative moment in slabs and girders of continuous spans, (*d*) Treatment of expansion and construction joints, (*e*) Paving, (*f*) Drainage and waterproofing of floors; arrangement of shear reinforcement to provide for higher unit values, which seem justifiable as disclosed by recent tests. These and many other details apparently elementary are in need of unification.

2. *Assumptions in Design.*—Included under this topic would be weights of all materials, earth pressures against retaining walls, allowable foundation pressures, assumptions in arch design, distribution of loads through earth fills and of concentrated loads on slabs, etc.

3. *General Specifications.*— To control selection of materials and regulate performance of work.

4. *Methods of Construction.*—To embrace such subjects as centering in arch design and form work for beam and slab construction, plant layout and recommendations covering the various methods of depositing concrete, etc.

The necessary preliminaries to the lasting solution of the committee work are to seek the unreserved coöperation of all state bridge engineers and others interested in the advancement of the work, to gather their opinions, seek their suggestions concerning differences encountered in practice and to collect typical plans and specifications in order to give actual measure of representative work. The state engineer is the important medium, for eventually all states will enact the Iowa law previously mentioned.

These preliminaries have been started and the interest shown is highly gratifying. Numerous state bridge engineers have sent constructive replies and considerable amount of valuable data with expressions of a willingness to coöperate with the committee to the limit of their experience.

The following specific questions were recommended for advisement of the present committee:

1. A review of highway load assumptions to be used in the design of concrete highway bridges and culverts.

2. The distribution of concentrated loads on simple slab superstructures having no earth covering.

3. The distribution of concentrated loads on floor slab and girders of through girder bridges having no earth covering.

4. The distribution of concentrated loads on the floor slab and beams of T-beam and deck girder bridges having no earth covering.

5. The distribution of concentrated loads through earth over-burden of various depths.

6. Design assumptions concerning the treatment of earth pressures on abutment and wing walls.

1.

REVIEW OF HIGHWAY LOAD ASSUMPTIONS.

Table 1 was prepared from the data and specifications sent by the various state bridge engineers and shows load assumptions used by each.

Many of the states are considering, and some have passed, laws to limit the weight of truck load fixed by the carrying capacity of the highways. It is reasonable to predict that this will nowhere exceed the effect of the weight of a 20-ton truck, 2/3 of the weight on the rear axle and 1/3 on the front axle, wheel base 11 ft. c. to c., and wheel spacing 6 ft. c. to c. of

tread. No concrete highway bridge should be designed for less than 15-ton truck of same dimensions or a uniform load of 125 lb. per sq. ft., disregarding an increased factor due to impact.

So far as the investigation of street-car loadings has progressed they indicate that the maximum loading for city bridges should not exceed a

TABLE 1.—EFFECTIVE WIDTHS UNDER TWO-POINT LOADING, LOADS 5 FT. APART ON CENTER LINE PARALLEL TO SUPPORTS.

Total Load on Two Points.	Slab 835.	Slab 934.
5,000	....	17'.26=107.9% span
10,000	....	15'.72= 98.2% span
15,000	....	16'.73=104.5% span
20,000	....	17'.5 =109.4% span
40,000	16'.86=105.4% span	....
80,000	22'.52=140.7% span	....

40-ton car, 19 ft. c. to c. of double-wheel truck, 4 ft. 6 in. c. to c. of wheels and 20 ft. 6 in. c. to c. of end wheels of adjoining cars entrain. This is equivalent to the loading of an average interurban car. It would seem advisable to add a live-load increment of 25 per cent for impact. Special cases of heavier interurban cars would have to be treated accordingly. The tendency in street-car design is to reduce the existing loading.

2.

THE DISTRIBUTION OF CONCENTRATED LOADS ON SIMPLE SUPERSTRUCTURES HAVING NO EARTH COVERING.

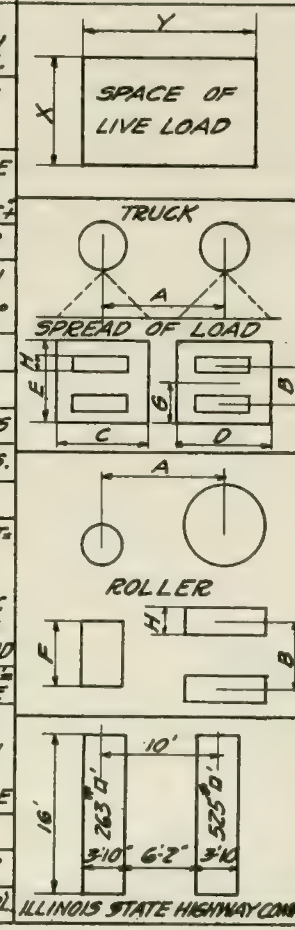
Previous to 1913 no experimental data was at hand upon which to base assumptions relating to the distribution of concentrated loads on simple reinforced-concrete slab superstructure having no earth covering.

In that year reports of tests were made by W. A. Slater before the American Concrete Institute and by A. T. Goldbeck before the Convention of the American Society for Testing Materials. Although these tests were small in scope they showed that a much wider distribution could be assumed than that practiced at the time and lead to further tests and developments. The data is now at hand, following which the engineer can formulate assumptions concerning this important factor in design with a reasonable assurance that full advantage can be taken of the inherent strength of the concrete.

In all these slab tests, the general procedure was to measure the deformations in the concrete or steel, or both, by means of a strain gage where the slab was loaded with a concentrated load. The resisting moments were taken as proportional to the area of the deformation curves. Effective widths were then calculated by equating the areas of the deformation curves as obtained from the strain-gage measurements to a rectangle

REGULATIONS GOVERNING HIGHWAY LOADINGS IN VARIOUS STATES

STATE	KIND OF LOAD	AXLE LOADS REAR	AXLE LOADS FRONT	IMPACT	A	B	C	D	E	F	G	H	X	Y	UNIFORM LOAD SPANS	UNIFORM LOAD LBS.#	REMARKS
CALIFORNIA	20 TON TRACTOR	28,000	12,000	NONE	12'0"	6'0"									50'-	150	SAME LOADING ON ARCHES, LOADS THRU EARTH ½ TO 1 SLOPE.
"	40,000 LBS.												8'0"	15'0"	50'+	60 ADD'L	
CONNECTICUT	20 TON ROLLER			25%											ALL	150	LOADS THRU EARTH SLOPE 45°
"	15 TON TRUCK			25%													
DELAWARE	20 " "	28,000	12,000	NONE	10'0"	5'0"						6" FRONT			ARCH	200	80# PER SQ.FT. ON SIDE WALKS. NO IMPACT IF FILL IS 12"+ OR SLAB 6"+
"	15 " "			25%*	FOR SLABS 6' OR LESS							14" REAR					
"	50 TON CAR	TRUCKS	30' C-C				SPREAD OF LOADS THRU FILL = 30°										TRUCK IN ADD'T. TO CAR
ILLINOIS	15 TON ROLLER	20,000	10,000	NONE											ALL	125	LOADS THRU EARTH
"	48,300# CONC'T.	32,200	16,100		10'0"		3'10"	3'10"	(SEE SKETCH)								UP TO 2'=20° 2'+=30°
IOWA	15 TON ROLLER	20,000	10,000		11'0"	6'0"		4'0" 5'0"				4'0" THIN SLABS* 6'0" HEAVY SLABS			ALL	100	*OVER STRINGERS
KENTUCKY	15 " "			25%													OVER 100' SPANS
MICHIGAN	18 " "	24,000	12,000	NONE	10'0"	5'0"				4'0"					100'-	100	REDUCE 1# IN 5' / 100# ON WALKS
NEBRASKA	20 " TRACTOR	26,600	13,400	NONE	11'6"	8'0"									100'-	100	MAIN STATE HIGH'YS.
"	15 " "	20,000	10,000	NONE	11'4"	7'0"									100'-	80	MINIMUM
OHIO	20 " "	26,600	13,400	NOTE	10'0"	6'0"	FOR INDUSTRIAL CENTERS						12'0"		ALL	150	WALKS 100# / 115-120 IMPACT = $\frac{S}{2}\left(\frac{100}{300+L}\right)$ S=MAX. STATIC L.L. STRESS L=LENGTH LOAD
"	15 " "	20,000	10,000	"	10'0"	6'0"	FOR FREQUENT MAX. LOADS						12'0"		"	120	" 80#
"	10 " "	13,300	6,700	"	10'0"	6'0"	FOR OCCAS. HEAVY LOADS						12'0"		"	100	
"	50 " CAR	4 WH. TRUCKS 32' C.C. 6' WHEEL BA.											12'0"		"	ADD'L	120#-150#
TEXAS	20 " ROLLER	28,000	12,000	NONE	10'0"	5'0"			10'0"		5'0"	1'8"			"	125	DISTRIB. THRU FILL OVER 1'=10' PLUS SLOPE 45° EA. SIDE
VIRGINIA	15 " TRUCK	20,000	10,000	25%x	10'0"	5'0"							9'0"	20'0"	"	100	15% IMPACT WHEN
"	12 " "	16,000	8,000	" x	10'0"	5'0"							9'0"	20'0"	ARCH	125	⊗ FILL IS FROM 2' TO 4'
"	6 " "	8,000	4,000	" x	10'0"	5'0"							9'0"	20'0"			NO IMPACT 4' OR MORE
WASHINGTON	20 " "	28,000	12,000	$\frac{100}{3P+200}$	12'0"	6'0"						1'8"			ALL	125	
W. VIRGINIA	15 " TRACTOR				10'0"	5'0"							12'0"	20'0"	ALL	80# ADD'L	DISTRIB. THRU FILL 30°
WISCONSIN	15 " ROLLER	20,000	10,000	NONE	10'0"	4'10"			9'0"	3'4"					40'-	125	REDUCE 3# FOR EA. 5' ADD'L



whose height equaled the maximum deformation and whose width was the "effective" width.

Very few experimental data have been more largely beneficial to reinforced-concrete design than these tests. Not alone for the light that they have thrown on the subject in particular nor for the economies effected, but for the unbounded confidence inspired by the knowledge of an unbelievable strength which can be interpolated and applied to similar problems wherein the monolithic action of the concrete is involved.

The wheel loads of a road roller or heavy truck are concentrated on small areas of a bridge floor. Heretofore the common practice in designing concrete slabs was to assume an effective width carrying these concentrated loads to be limited by the width of the tire plus a spread in width equal to twice the effective depth of the slab. As a result of these tests assumptions can now be made for an exceedingly wider distribution over an "effective" width of 0.7 of the span length, this factor being dependent upon the ratio of the width of the slab to the span length as will be shown later.

By assuming the effective width as a constant proportion to the span, the live-load moment per unit width of the slab is the same for a given load

TABLE 2.—DATA ON SLABS TESTED.

Slab No.	Dimensions, Ft.		Depth.		Steel Percentage.		Central Load Effective. Width—Span.	Failure.	
	Span.	Breadth.	Total.	Effective.	Long.	Trans.		Span.	Central Load.
835	16	32	12	$10\frac{1}{2}$	0.75	0.0	See next table.	16	119,000
930	16	32	10	$8\frac{1}{2}$	0.75	0.0		16	80,000
934	16	32	7	6	0.75	0.0		16	40,000

regardless of the span, but this is altered slightly since the effective width decreases slightly as the span increases and as the width of the slab affects the distribution. Variation of course exists in the total moment with variable span lengths owing to the constantly increasing increment due to the dead load.

The conclusion has been made that the effective width is affected very little by the percentage of transverse reinforcement (parallel with the supports). Whatever value has been found in the function of the transverse steel to further distribute the load, it is so small compared with amount necessary to show increased distribution as to make its use uneconomical. As a general consideration, however, transverse steel should be used in all reinforced-concrete slabs for the purpose of preventing displacement from the theoretical position during the pouring of the slab by wiring together with the longitudinal steel, and for the ultimate purpose of distribution deformations due to variations in temperature. The amount would vary, depending upon the length of the section to be poured in continuous operation and the cross-sectional area of the slab.

Tables 2, 3 and 4 are compiled from tests made under the direction of A. T. Goldbeck for the Bureau of Public Roads.

In Table 3 the effective widths are seen to vary with the load to some extent as in the case of Slab 934. The effective width, however, varies from 0.7 to 0.8 of the span for working loads.

Conclusions.—The above effective widths are based on concrete deformations rather than on steel deformations, since the concrete gave the smallest values for effective width. Comparing two-point with single-point loading, it would seem that the increase in effective widths due to two-point loading is somewhat less than the distance between the loads.

TABLE 3.—EFFECTIVE WIDTHS UNDER CENTRAL LOADS.

Center Load.	Slab 835 10½ in. Effective Depth.	Slab 930 8½ in. Effective Depth.	Slab 934 6 in. Effective Depth.
15,000	....	11'.4=71.6% span	12'.7=79.5% span
20,000	11'.6=72.3% span	13'.0=81.2% span	17'.5=109.3% span
25,000	11'.5=71.9% span	12 .9=81.1% span	....
32,500	12.'1=75.7% span	....	....
35,000	....	14'.5=90.7% span	....
Failure	119,000 lb.	80,000 lb.	40,000 lb.

TABLE 4.—DATA ON EFFECTIVE WIDTHS (SEE FIG. 1).

Total Width Divided by Span.	Effective Width Divided by Span.
0.1	0.1
0.2	0.2
0.3	0.28
0.4	0.37
0.5	0.44
0.6	0.5
0.7	0.55
0.8	0.58
0.9	0.62
1.0	0.65
1.1	0.67
1.2	0.68
1.3	0.70
1.4	0.71
1.5	0.72
1.6	0.72
1.7	0.72
1.8	0.72
1.9	0.72
2.0	0.72

To determine what effect the ratio of the span width to span length had on the "effective width" of distribution a number of slabs were tested and were then decreased in width several times by cracking off their ends. The results of these tests are best seen in the diagram, Fig. 1. In order to be conservative the curve showing the variation of effective width with the total width was drawn through the lower side of the zone of results.

Table 4, taken from the curve, shows how the "effective width" varies with total width in slabs loaded with single, central, concentrated load.

An eccentric load, such, for instance, as occurs when a heavy truck traverses the side of a bridge, or a roller backs up to the parapet to take on water, may control the design. To investigate this, a slab (A-25) of 16-ft. span and 72-ft. width, 13-in. effective depth, 0.75 per cent longitudinal steel, no traverse steel, was tested. Fig. 2 best shows the results obtained. The curve of long dashes is based on the results of previous tests and is plotted from Table 4, showing the effect of total width on effective width. The solid curve represents the value for effective width obtained in the test of this particular slab (A-25) when loaded eccentrically as shown. The dotted curve is plotted by assuming that the effective width equals the distance between the load and the nearest side of the slab plus half the effective width of the same slab when centrally loaded, the values of which are

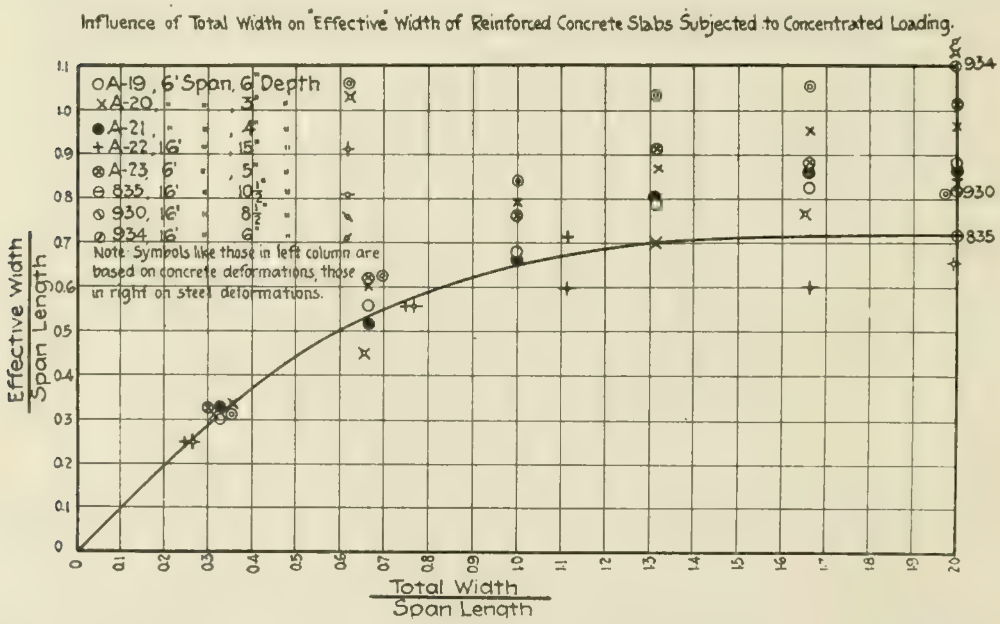


FIG. 1.—INFLUENCE OF TOTAL WIDTH ON "EFFECTIVE" WIDTH OF REINFORCED-CONCRETE SLABS SUBJECTED TO CONCENTRATED LOADING.
[From Tests by A. T. Goldbeck for Bureau of Public Roads.]

plotted in the curve of dashes. As the values of the effective width calculated in this way so nearly equal the measured values, the conclusion reached was as follows:

Conclusion.—When the load is nearer to the edge of the slab than half the effective width of the same slab centrally loaded, the effective width then equals the distance from the load to the side of the slab plus half the effective width under central loading.

$$b_e = D + \frac{b_c}{2}$$

where b_e = effective width of the slab eccentrically loaded.
D = distance from load to nearest side.
b_c = effective width of the slab centrally loaded.

The value of b_c for any case may be obtained from Table 4 or Fig. 1.

This conclusion is obtained from the test of one slab and so may not be strictly general, although it is probably approximately correct.

In order to strengthen the slab at the sides to take care of the eccentric load, the curb of the parapet wall should be designed to supply the deficiency in strength between a centrally loaded and an eccentrically loaded slab.

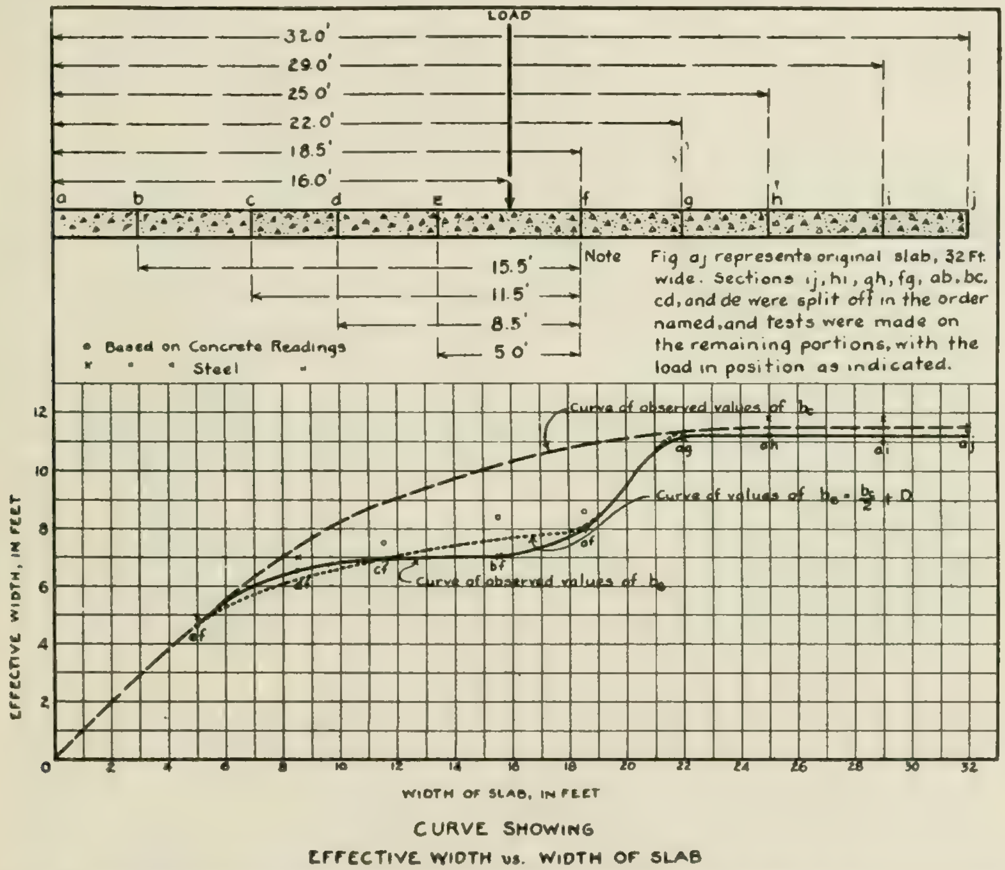


FIG. 2.—CURVE SHOWING EFFECTIVE WIDTH VS. WIDTH OF SLAB IN REINFORCED-CONCRETE BRIDGE SLABS.

RECOMMENDATIONS FOR THE DESIGN OF REINFORCED-CONCRETE SLABS SUBJECTED DIRECTLY TO CONCENTRATED LOADS.

Use narrow rectangular beam theory, substituting for the breadth the effective width as shown in Table 4.

Double Central Loads Spaced 5 ft. Apart.—(Use above values for effective width plus 4 ft.)

Single Eccentric Load.—(*a*) Effective width equals half of values in Table No. 4, plus distances to nearest side where this distance is less than half effective width; (*b*) when distance between load and nearest side is more than half of effective width given in above table, use the above values for effective widths. When double central loads are spaced 5 ft. apart the

parapet at the side of the slab may be designed to supply the deficiency in the resisting moment, due to the difference in effective widths of the centrally and eccentrically loaded slabs.

3.

DISTRIBUTION OF CONCENTRATED LOADS ON THE FLOOR SLAB AND GIRDERS OF A THROUGH-GIRDER BRIDGE HAVING NO EARTH COVERING.

The distribution of concentrated loads on the floor slab of a reinforced-concrete highway through-girder bridge should be made in the same manner as recommended for simple slab superstructures.

The present practice in the determination of the girder section is to place the truck or roller loading as close as possible to the girder in position, giving maximum moment for the span and consider the wheel loads as concentrated.

That this is very conservative is revealed by a test on a 40-ft. through-girder bridge 17 ft. in clear width, as reported by Prof. D. A. Abrams in the 1913 Proceedings of the A. S. T. M. He found the actual measured stresses in the steel due to a uniform loading on the slab to be about one-half of the computed stress. This was no doubt due partially to the tension in the concrete, but for the most part to the added stiffness given to the girder by the floor slab.

Further consideration of this factor will unquestionably lead to the assumption of distributing the wheel concentrations over some appreciable length of girder in a uniform loading.

4.

THE DISTRIBUTION OF CONCENTRATED LOADS ON THE FLOOR SLAB AND T-BEAMS OF DECK-GIRDER CONSTRUCTION HAVING NO EARTH COVERING.

No tests dealing with the distribution of a concentrated load over a series of reinforced-concrete T-beams have reached the committee. However, the Highway Department of the State of Ohio has performed a series of tests, printed in their Bulletin No. 28, in determination of the distribution of a concentrated load by a reinforced-concrete slab superimposed upon a series of I-beams, the I-beam corresponding to the stringers of a steel bridge. From these tests the following conclusion has been drawn: "When one or more concentrated loads not closer than 6 ft. rest upon a floor which is formed of joists and beams approximately uniformly spaced, and all loads are midway between beam ends of supports, the greatest weight that can fall upon any one beam may be assumed equal to the weight of one load multiplied by the ratio S/F, where S is the beam spacing in feet and F is 6 for a concrete slab. The factor F varies for other types of floor, being 4 for a 3-in. plank floor and 5 for both wood and block floor on 3-in. plank and a 4-in. strip floor.

For the reinforced-concrete beam type in highway bridge construction it rarely occurs that a spacing less than the tread of a truck, 5 to 6 ft., is warranted, particularly for the longer spans. The load carried by this beam spacing should not be less than the weight of one wheel load.

In considering the slab of T-beam construction alone, which theoretically would require no reinforcement, the distribution of concentrations may be treated in a manner similar to that of simple slabs. The span of the slab can be taken as the clear distance between T-beams and sufficient reinforcement placed to take the maximum bending at the center of the span. The bars, however, need not be bent up over each beam, as the com-

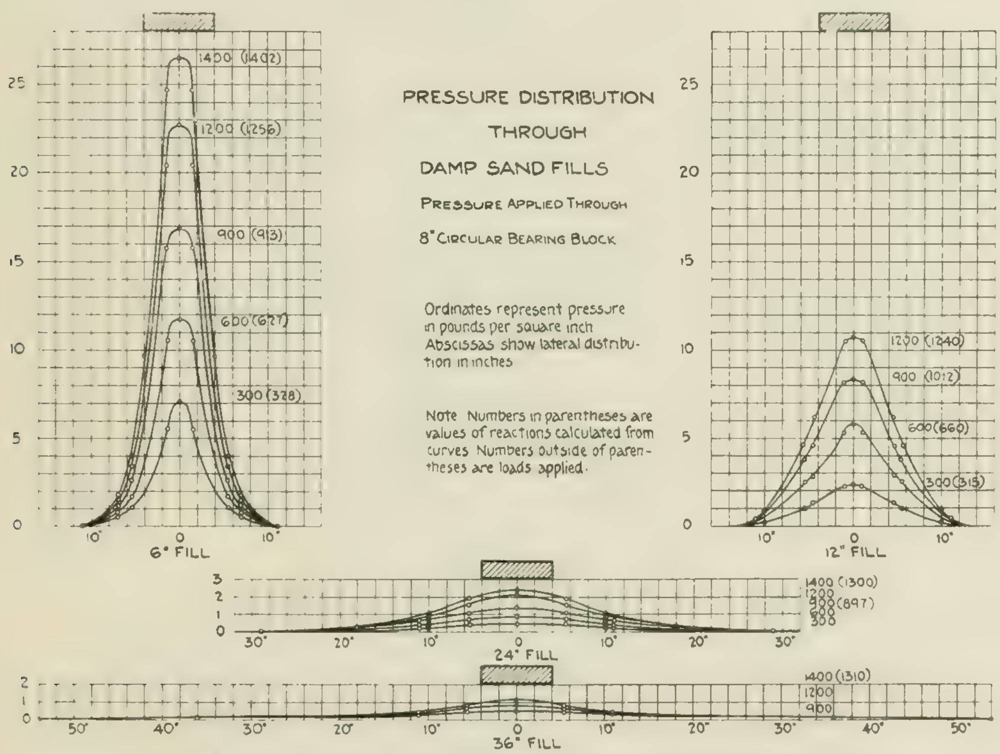


FIG. 3.—PRESSURE DISTRIBUTION THROUGH DAMP FILLS.

pression of the T will more than offset the tension of the slab due to continuity of the spans.

A considerable amount of arch action is thrown into the flange of the T-beam, where the tension is reversed to the upper plane over the support of beams in continuity. So that, here, too, a reversal of stress in the slab, due to a concentrated load on it, is precluded.

5.

DISTRIBUTION OF CONCENTRATED LOADS THROUGH EARTH OVERBURDEN OF VARIOUS DEPTHS.

The results obtained in tests performed by Bureau of Public Roads on sand fills from 6 in. to 5 ft. deep, have been published in 1917 Proceedings

of A. S. T. M. The apparatus used consisted of small cells buried in the earth, with an indicating instrument conveniently placed for reading. Air pressure in the cell equilibrates soil pressure on the cell as determined by breaking of electrical contact, and the air pressure, which then equals soil pressure, is read on a pressure gage. Loads were applied up to 5,000 lb. through bearing blocks 8 in. and 13½ in. in diameter. Typical pressure

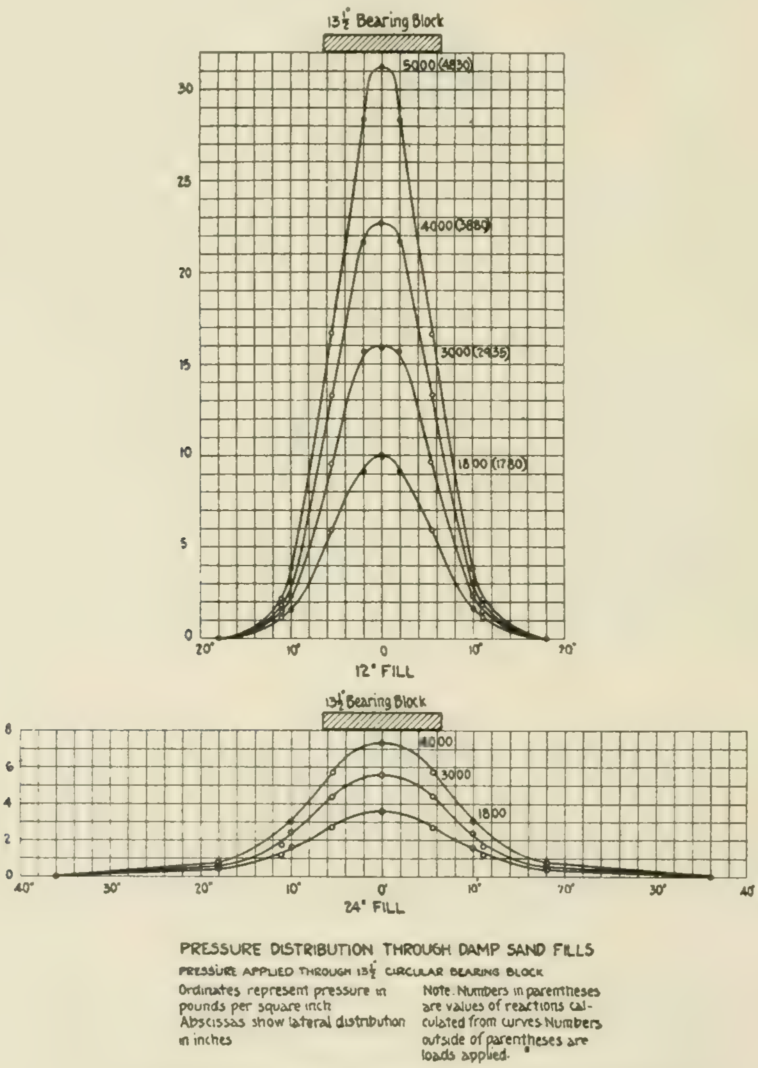


FIG. 4.—PRESSURE DISTRIBUTION THROUGH DAMP FILLS.

curves are shown in Figs. 3, 4, 5 and 6. It will be noted that for fills 24 in. or less in depth, that most of the load is transmitted to the slab over an area not much greater than that of the bearing blocks on top of the fill, whereas, in the case of the 5-ft. fill, there is, for practical purposes, a uniform distribution of the pressure over the slab.

No rules for the design of slabs having an earth fill, and subjected to a concentrated load, can be formulated on the basis of the tests thus far made, but apparently, for fills of sandy material less than 2 ft. in depth,

no further distribution of a concentrated loading than given for direct bearing on slab previously recommended would be warranted. Fig. 6 will show the designer the spreading effect of fill up to 48 in. depth.

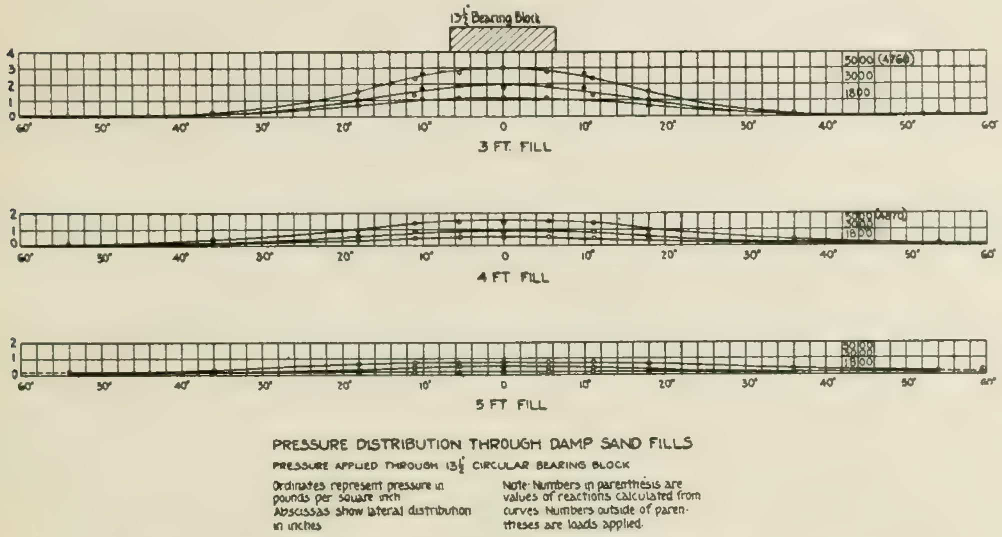


FIG. 5.—PRESSURE DISTRIBUTION THROUGH DAMP FILLS.

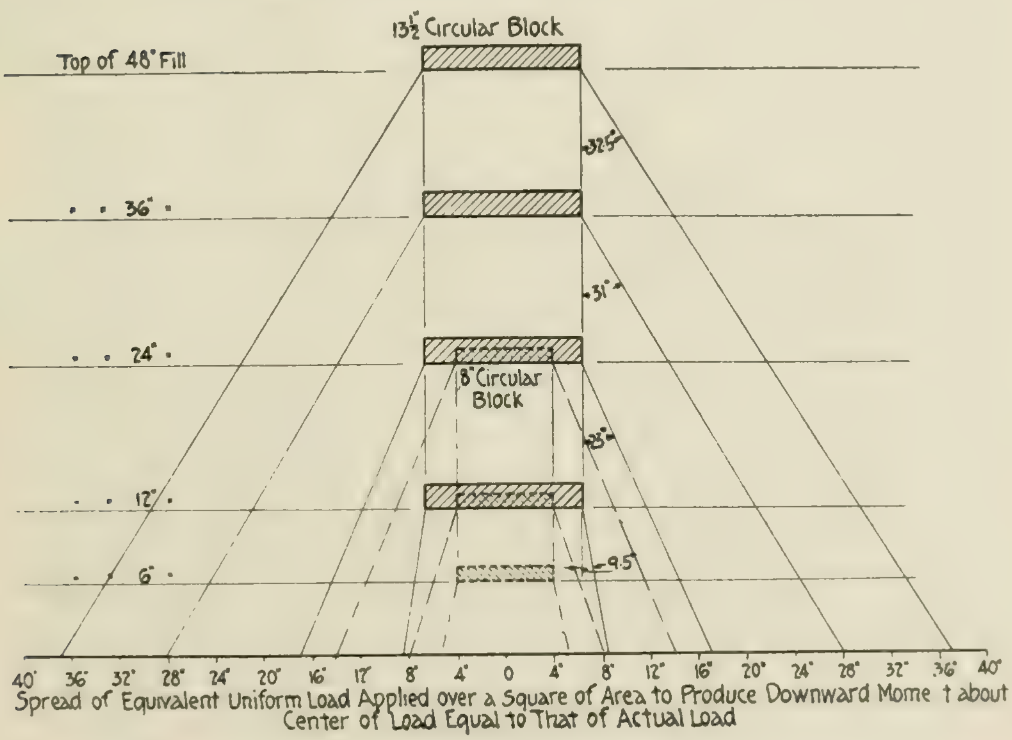


FIG. 6.—SPREAD OF EQUIVALENT LOAD.

It is hoped that these and other tests will be enlarged upon to show the spreading effect of concentrated loads, shaped and spaced like wheels and rollers, on several other kinds of material supported on concrete slabs.

It is the general practice to spread loads sidewise through deep fills on a slope of about ½ to 1 from the edge of the concentration, and in the other direction by distance, center to center, of concentrations, if these are entrain, such as wheel loadings.

6.

DESIGN ASSUMPTIONS CONCERNING THE TREATMENT OF EARTH PRESSURE ON ABUTMENT AND WING WALLS.

There was not sufficient time to collect the data on this subject for a résumé at this time, but it should by all means be given a thorough consideration by the committee for the next report. The committee has been promised some very valuable data covering this important consideration.

COMMITTEE ON HIGHWAY BRIDGES AND CULVERTS,
A. B. COHEN, *Chairman.*

REPORT OF COMMITTEE ON NOMENCLATURE.

The Committee on Nomenclature has continued active during the past year. Most of the work has been carried on by correspondence owing to the wide separation of the members. One meeting of the Committee was held at which three members were present.

The committee's work has included the preparation of definitions of words commonly used in the concrete industry, and the adoption of standard forms for use in Institute publications.

In the work of preparing definitions, the question arises as to whether the committee should consider as its function the endorsement of the forms used in the best practice or the forms which it believes logical and correct. Obviously, neither standard can be adopted to the exclusion of the other. The former would lead to conflict as to what is the best practice and to confusion, and the latter would lead to antagonism of other interests whose co-operation the American Concrete Institute needs.

For terms falling within the committee's scope, the policy has been:

(1) To make the definitions fundamental and as broad as possible, with recommendations as to expansion to fit special needs and notation as to the most common usage, but to avoid needless restriction of the meaning of the word which would make it apply to only one or more phases of a larger possible meaning.

(2) To adopt as far as possible the definitions already accepted by other societies, authors, and by standard practice in engineering.

The 1917 report of the Committee on Nomenclature forms the basis of the definitions included in this report. The definitions have been modified by the committee in many respects as a result of criticisms received in fifty-two replies to a questionnaire sent out in 1917 to members of the Institute and others.

The standard forms included under the headings "Abbreviations," "Numerals," "Spelling and Punctuation," "Capitals," "Standard Terms and Forms of Expression," and "Footnotes," have been taken from the "Standards" of the American Society for Testing Materials.

The standard "Symbols" are taken from the Final Report of the Joint Committee on Concrete and Reinforced Concrete.

The standard forms have been adopted not with the view of anticipating all possible questions that may arise, but of providing for consistency of practice in matters of common occurrence.

I. AGGREGATE.

1. *Aggregate.*—The inert material used in making concrete.

(*a*) *Fine Aggregate.*—The finer inert material used in making concrete, usually considered to include that material passing a sieve having four meshes per linear inch.

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2. *Sand.* The finely divided material, generally of a silicious nature, resulting from the reduction of rock by natural forces to the size included under fine aggregate.

3. *Stone Screenings.*—Crushed natural rock of sizes defined under "Fine Aggregate."

4. *Coarse Aggregate.*—The coarser inert material used in making concerte, usually considered to include that material which is retained on a sieve having four meshes per linear inch. The upper limit of its size depends on various conditions but it seldom exceeds three inches.

5. *Crushed Stone.*—Crushed natural rock of sizes defined under "Coarse Aggregate."

6. *Crushed Slag.*—Air-cooled blast-furnace slag of sizes included under "Coarse Aggregate."

7. *Cinders.*—The hard waste product of the combustion of coal.

8. *Plums.*—Stones of large size, usually over 3 in., which are added to the concrete during placing.

9. *Gravel, Bank-Run Gravel.*—Normal product of a gravel bank including pebbles and sand in varying proportions.

10. *Crushed-Run Rock.*—The unscreened output of the stone crusher.

11. *Chats.*—A crushed rock of flinty character obtained as a by-product in the preparation of lead and zinc ores for smelting. It ranges in size from about $\frac{1}{4}$ in. to $\frac{3}{8}$ in.

11-A. *Slag Screenings.*—Crushed air-cooled blast-furnace slag of sizes defined under "Fine Aggregate."

11-B. *Pebbles, Screened Gravel.*—The material obtained by screening gravel to sizes defined under "Coarse Aggregate."

II. CEMENT.

1. *Portland Cement.*—The product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

2. *Natural Cement.*—The finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature sufficient only to drive off the carbonic acid gas.

3. *Sand Cement, Silica Cement.*—The finely pulverized product resulting from the intimate mixing and grinding in varying proportions (generally one-half of each) of silicious sand or other silicious material and portland cement.

4. *Puzzolan Cement.*—The finely pulverized product of a mechanical mixture of volcanic ashes or basic blast-furnace slag with powdered slaked lime. When slag is used this is sometimes known as slag-cement. This should not be confused with portland cement manufactured from a mixture of slag and limestone but calcined subsequent to mixing.

III. CONCRETE.

1. *Concrete.*—A compound of gravel, broken rock or other aggregate united by means by hydraulic cement, coal tar, asphaltum, or other cementing material. General when a qualifying term is not used, "portland cement concrete," is understood.

2. *Grout* (noun).—The material resulting from mixing cement and water, or cement, sand and water, in fluid consistency .

3. *Grout* (verb).—To fill a cavity or make a joint with grout.

4. *Mortar.*—A material used in a plastic state, becoming hard in place, to bond together such materials as brick, stone, tile, gypsum blocks, terra cotta, etc., in building walls, partitions, columns, foundations, piers, floors, and roof arches, etc. The word "mortar" is used without regard to the composition of the material, defining its use as a binding material, as contrasted with the words, "stucco" and "plaster."

5. *Precast Concrete.*—Concrete cast in separate units which are later assembled into a structure.

6. *Reinforced Concrete.*—Concrete in which metal (generally steel) has been embedded in such a manner that the metal and the concrete assist each other in taking stress.

7. *Rubble Stone or Cyclopean Concrete.*—Concrete in which large stones (plums) are embedded after mixing and during placing.

IV. CONSTRUCTION.

1. *Book Tile.*—A tile designed to be held in place by means of crude tongues and grooves formed on the edges of the tile and deriving its name from its resemblance to a book.

2. *Screed.*—A guide (usually of wood) used to gage the finishing of mortar or concrete to required surface or grade.

3. *Sleeper.*—A strip of wood placed above or in a concrete slab to serve as a nailing strip for wood flooring.

4. *Sleeve.*—A tube or box of wood or metal placed on or in the forms of a concrete floor or wall in order to provide a hole for the passage of a pipe, bolt or wire, or to act as a spacer for the forms.

A short length of pipe used to hold in alignment the butted ends of compression reinforcing bars.

5. *Spouting Concrete.*—The transporting of concrete from the place of mixing by gravity through troughs or tubes.

6. *Unit Construction.*—The utilization of precast concrete members in building construction. (See definition—*Precast Concrete.*)

V. ENGINEERING DESIGN AND THEORY.

1. *Belt Course.*—A continuous horizontal course in the outside wall of a building projecting slightly from the elevation and usually molded in order to produce an architectural effect.

2. *Column Capital.*—An enlargement of the upper end of a column generally used in connection with flat-slab floors.

3. *Column Head.*—(See *Column Capital.*)

4. *Continuous Action.*—The working together of two or more flexural members (columns, beams, or slabs) to resist mutually a moment applied to one or more of the members.

5. *Curtain Wall.*—A wall which does not itself form a supporting member of the building of which it is a part.

6. *Dropped Panel.*—The portion of a flat-slab around the column capital which lies below the plane of the lower surface of the remainder of the floor slab.

7. *Flat-Slab Floor.*—A flat concrete floor having reinforcing rods extending in two or more directions and having no beams or girders to carry the loads to the columns. Various trade names are applied to flat-slab floors using proprietary systems of reinforcement.

8. *Negative Reinforcement.*—Reinforcement so placed as to take stress caused by negative bending moment.

9. *Punching Shear.*—The shear around the periphery of a strut or other member applying a concentrated load. Punching shear may be critical in slabs or other relatively thin members.

10. *Wall Beam.*—A beam which spans between exterior columns of a structure and which is used as a supporting member for the floor slab at its outside edge. Frequently the wall beam carries the curtain wall for the story above.

11. *Weep Hole.*—A hole in a wall, floor or other structure made for the purpose of provided drainage.

12. *Well.*—A vertical compartment or shaft in a building or a series of openings in vertical alignment through the floors of a building: "Elevator-well" when used for operating an elevator; "stair-well" when used to enclose a stairway.

VI. FINISH.

1. *Cement Wash.*—A coating consisting of a mixture of cement and water applied to concrete surfaces (generally with a brush) to reduce permeability, or to give a uniform color.

2. *Composition Flooring.*—A floor formed of various chemicals and a filler mixed with water and laid in a plastic condition, troweled smooth and then allowed to set hard.

3. *Granolithic Finish.*—A finish applied to concrete floors which exposes to view a surface layer of particles of coarse granite, or of other hard enduring rock bonded with cement "mortar." The finish may be formed by troweling while wet to bring the crushed rock to the surface or by surface grinding after the concrete has hardened. Apparently, the name has been derived from the use of the crushed granite, in the surface layer.

4. *Plaster.*—A material used in a plastic state to form a hard covering for the interior surface, walls, ceilings, etc., of any building or structure. The word "plaster" is used without regard to the composition of the material, defining only its use and location of use as contrasted with the words "stucco" and "mortar."

5. *Stucco.*—A material used in a plastic state to form a hard covering for the exterior walls or other exterior surfaces of any building or structure. The word "stucco" is used without regard to the composition of the material, defining only its use and location of its use, as contrasted with the words "plaster" and "mortar."

6. *Tooling.*—The finishing of a concrete surface with a special tool in a manner to show the marks of the tool.

VII. TERMS USED IN CONNECTION WITH FORMWORK.

1. *Form.*—A structure (or a structural unit which in conjunction with other units makes up a structure) to receive the concrete before it has hardened and mold it to the designed shape. Also known as falsework or centering.

2. *Bottom Form.*—The form unit used to define the bottom of a structural member as a beam or girder.

3. *Side Form.*—The form unit used to define the side of a structural member as a beam or girder.

4. *Form Clamp.*—An arrangement of wood, steel or iron members designed to hold forms together to resist the pressure of wet concrete against their sides.

5. *Form Girt.*—A longitudinal timber laid cross-wise under the joists and used as a girder to support the floor formwork.

6. *Form Jack.*—A vertical support for the formwork of beams which is provided with a cross-piece at the top and two short diagonal braces.

7. *Form Joist.*—A longitudinal timber placed under the panels of floor formwork.

8. *Form Ledger.*—A horizontal timber used to brace the posts in floor formwork.

9. *Form Mud Sill.*—A timber member laid upon the surface of the rough ground to provide a bearing for the posts of the floor formwork.

10. *Form Panel.*—A form unit used to define a flat surface of considerable extent, *e. g.*, a slab "bottom form" or a wall "side form."

11. *Form Post.*—A vertical timber used to support the formwork of floors.

12. *Shore.*—A prop or support. A temporary vertical timber used to support the weight of a floor after the forms are stripped and until the concrete has thoroughly hardened.

13. *Form Re-Stud.*—(See "*Shore.*")

14. *Form Ribband.*—A horizontal timber used to hold the bottoms of beams side-forms in place.

15. *Angle Fillet.*—A triangular strip of wood which is placed in the angle of a form for concrete in order to produce a chamfered edge in the concrete.

16. *Handle Nut.*—A nut with a handle forged on same to obviate the necessity for using wrenches; much used in form work and other temporary construction.

17. *Insert.*—A socket usually of cast or malleable iron, either slotted or

ABBREVIATIONS (*Continued*).

(*e*) *Units of Time.*

Afternoon	p. m.
Day	spell out
Forenoon	a. m.
Hour	hr.
Minute	min.
Month	spell out
Second	sec.
Week	spell out
Year	spell out

(*f*) *Electrical and Magnetic Terms.*

Ampere	spell out
Electric horse power	e. h. p.
Electromotive force	e. m. f.
Magnetomotive force	m. m. f.
Ohm	spell out
Volt	spell out

(*g*) *Units of Power.*

Brake horse power	b. h. p.
Horse power	h. p.
Indicated horse power	i. h. p.
Kilowatt	kw.
Watt	spell out

(*h*) *Units of Heat.*

British thermal unit	B. t. u.
Calorie	cal.
Centigrade*	C.
Degree*	°
Fahrenheit*	F.

(*i*) *Miscellaneous Technical Terms.*

Birmingham wire gage	B. w. g.
Browne & Sharpe (gage)	B. & S.
Chemically pure	c. p.
Degree (angular measure)	deg.
Diameter	spell out
Revolutions per minute	r. p. m.
Specific gravity	sp. gr.
Tensile strength	tens. str.
United States (gage)	U. S.

(*j*) *Miscellaneous General Terms*

Figure	Fig.
Number	No.
Per	spell out
Per centum	per cent
Proceedings	Proc.
Plate	spell out
Table	spell out
Transactions	Trans.
Volume	Vol.

(*k*) Use abbreviations only after nouns denoting a definite quantity, except in tabular work. For example: "The tensile strength is 45,000 *lb. per sq. in.*"; but "The tensile strength in *pounds per square inch* is ....."

(*l*) When terms are used in an abstract or descriptive sense, they shall not be abbreviated. For example, use "the *magnetomotive force* is applied"; not, "the m.m.f. is applied."

(*m*) Use a period after each abbreviation, except after *per cent*, and as noted in Paragraph (*o*).

(*n*) All abbreviations shall be used in the singular. Thus, "two inches" shall be abbreviated "2 in."; not "2 ins."

(*o*) *Compound Words.*—The abbreviations for compound words, when used, shall be formed by connecting the abbreviations of the separate words by a hyphen, omitting the period preceding the hyphen. Thus, "ft-lb., watt-hr., kw-hr., m-kg.," etc.

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* See paragraph (*r*), p. 381.

(*p*) *Symbols.*—Avoid the use of symbols. Do not use (′) or (″) in either text or tables; their use is permissible in illustrations. The symbol (%) shall not be used in the text, but may be used in tables when lack of space requires it. See Paragraph (*r*).

(*q*) The word "percentage" shall be used when not following a number. Thus, "the *percentage* of carbon shall be"; not, "the *per cent* of carbon shall be." But, "0.35 *per cent* of carbon."

(*r*) After numerals, use the following abbreviations: "62° F., 36° C." In expressions like the following, omit the degree mark after the first figure: "75 to 80° C." In a table heading, use "Temperature, *deg. Fahr.*," or "*deg. Cent.*"

(*s*) In expressing dimensions, use the following form: "2 *by* 4 in. in section;" not "2 x 4 in. in section," nor "2 in. by 4 in. in section."

(*t*) Spell out the names of the months: as, "January 25." Do not use the form "January 25*th*."

(*u*) In text, do not abbreviate "namely" and "that is."

(*v*) Spell out names of companies, railroads, etc., using the ampersand (&) only between proper names. Abbreviate "Company" in firm names. For example: "Brown & Sharpe Manufacturing Co.," "Philadelphia & Reading Railway Co."; but, "American Steel *and* Wire Co."

(*w*) In giving a title, use Dr., Prof., Genl., etc., where initials or full name is given; spell out where surname only is given.

NUMERALS.

(*a*) Roman numerals will be used in designating tables and plates: thus, "Table VI"; not "Table 6." Arabic numerals will be used in designating figures: thus, "Fig. 3"; not "Fig. III."

(*b*) Spell out all numbers from one to twelve, with the following exceptions:

1. Use numerals when the quantity is partly or wholly fractional: as, 1.15, $1\frac{1}{2}$, $\frac{1}{3}$.

2. Use numerals when followed by an expression having a standard abbreviation: as, 1 in., 6 lb., etc.; except where the statement is vague in nature, in which case neither numerals nor abbreviations shall be used: as, "about *six pounds*," etc.

3. If for any reason the standard abbreviation of the expression following the number is not used, or if the expression does not admit of abbreviation (as *ohm*, *ton*, etc.) the use of numerals shall be optional, unless covered in the following paragraphs.

4. In contrasted statements, if some numbers must be numerals, use numerals for all: as, "2 miles and 16 miles."

5. In a series of connected numerical statements implying precision, use numerals: as, "2 years, 5 months, 3 days." The use of numerals (especially the "1") is not recommended for numbers occurring in precise statements similar to the following: "By connecting the *two* test coils;" shall consist of *two* equal and uniformly wound solenoids," etc.

6. Use numerals after abbreviations: as, Vol. 6, Fig. 2, etc.

(*c*) Use numerals for all numbers exceeding twelve, with the following exceptions:

1. Do not begin a sentence with a numeral.
2. Round numbers used in an indefinite sense shall be spelled out: as, "A *hundred* feet or so," etc.
3. Numbers shall be spelled out when used in the following manner: *fifteen* 2-in. rods," etc.

(*d*) In expressing percentages, precise figures, etc., use decimals: as, "4.5 per cent"; not "4½ per cent."

(*e*) In decimal numbers having no units, a cipher shall be placed before the decimal point: as, "0.65 in."; not ".65 in."

(*f*) Omit unnecessary ciphers in sums of money: as, "$3"; not "$3.00."

(*g*) In pointing off numbers of more than four figures, use commas in the text (1,234,567) and spaces in tabular matter (1 234 567). Numbers of four figures shall not be pointed off in either text or tabular matter (1234), except when they occur in a table containing any number of more than four figures.

(*h*) Always use numerals for the day of the month when the month is given (January 25, 1913) and for the time of day (2.30 p. m.).

SPELLING AND PUNCTUATION.

(*a*) *Simple Words.*—The following spelling shall be used:

aging	fulfil	oxide
briquette	gage	paraffin
cent*er*	gasol*i*ne	program
crystallin	glycerin	re*i*nforced
dis*k*	iodin*e*	skil*l*ful
*e*mbed	*i*nsure	sul*f*ur
fib*er*	mold	turpentine
formula*s*		

(*b*) *Compound Words.*—The following spelling shall be used:

Spell with hyphen.

cold-roll	re-anneal
cross-section	re-treat
one-half	rough-forge
open-hearth	stop-cock

Spell without hyphen when used as noun.

cast iron	plaster of Paris
crank shaft	testing machine
drop test	water bath
engine bolt	wrought iron
piston rod	

Spell as one word.

cooperate	reheat
eyebar	reroll
firebox	retest
fireproof	reweigh
footnote	sinkhead
limewater	staybolt
quicklime	

(*c*) Compound adjectives shall be hyphenated; as "2-in. gage," "cast-iron cylinder," "500-horse-power (or 500-h.-p.) motor," "0.20-per-cent-carbon steel," etc. Such expressions as the following shall be written *without* the hyphen after the first numeral: "2 and 6-in. specimens."

(*d*) Do not hyphenate such expressions as "newly puddled iron," where the adverb is a regular modifier of the adjective.

CAPITALS.

(*a*) Use capitals sparingly.

(*b*) Capitalize the principal words in headings, titles of books, papers, etc. (nouns, verbs, adjectives and adverbs).

(*c*) Use capital initial "C" for "committee" when used as a title: thus, "Committee A-1," "Committee on Papers." In all other cases use lower-case "c"; thus, "The committee recommends ......"

(*d*) Use capital initial "B" for Bessemer; "P" for Portland.

(*e*) Use initial capitals in reference to volumes, figures, plates, etc.: as, Vol. 6, Fig. 2, Plate VI, Table III.

(*f*) Use the form "test No. 1," "specimen A," etc.

STANDARD TERMS AND FORMS OF EXPRESSION.

(*a*) The numbered sections of a standard shall be referred to as "Section 6"; the lettered sub-divisions of a section shall be referred to either as "Paragraph (*a*)," or "Section 6 (*a*)." The former shall be used only when the reference occurs in the section containing the paragraph referred to; in all other cases the latter form shall be used.

(*b*) Use "shall" wherever the standards are to be made binding on parties of the first or second part.

(*c*) Use "will" wherever the standards are intended to express a declaration of purpose not mandatory upon the parties of the first or second part.

(*d*) Use "may" wherever the standards provide definitely for alternative courses.

(*e*) Use "full-size tests"; not "full-size*d* tests," etc.

(*f*) Use "gage length"; not "gage*d* length."

(*g*) Use "test *specimen*"; not "test *piece*." In case the term "test specimen" is repeated several times in the same section, the word "specimen" may be used after the first use of "test specimen."

(*h*) Use "⅜ in. *or* over in thickness"; not "⅜ in. *and* over."

(*i*) In referring to dimensions, use simply "2 in."; not "two inches (2 in.)," or "two (2) inches."

(*j*) Use the form "without *cracking*" in referring to bend tests of metals; not "without *sign of cracking*."

(*k*) Use "melt" to mean "melt of steel," "blow of steel," and "heat of steel," as distinguished from "treating-plant heat, "etc.

(*l*) Use "reduction *of* area"; not "reduction *in* area" or "*contraction* in area."

(*m*) Use "acid *number*," "iodine *number*," etc.; not "acid *value*," "iodine *value*," etc.

FOOTNOTES.

a) Use superior figures instead of asterisks, etc., except in connection with numerals, for which use letters.

(*b*) The names of journals, proceedings, bulletins, etc., shall be printed in italics, without quotation marks; the titles of papers and reports shall be printed in Roman and enclosed in quotation marks.

(*c*) Abbreviate the names of societies.

(*d*) When reference is made to a paper or report by title, only the initial page shall appear in the footnote. Thus:

(Name of Author) , " (Title of Paper) ," *Proceedings*, Am. Soc. Test. Mats., Vol. XIII, p. 450 (1913).

When such titles are not given, or when reference is made to certain parts of papers, reports, etc., page numbers shall be indicated, as follows:

Railway Age Gazette, Feb. 16, 1912, pp. 280–282.

(*e*) When volume numbers are given, the year of publication shall appear in parentheses at the end of the footnote. Otherwise, the date of publication shall appear immediately after the name of publication. (See above examples.)

(*f*) Footnotes shall always appear at the bottom of the page on which the reference is made.

(*g*) References, by the use of such terms as *ibid.*, etc., to previous footnotes, shall not be used unless the footnote referred to is given on the same page; otherwise the footnote in question shall be repeated.

SYMBOLS.

(*a*) *Rectangular Beams.*

The following notation is recommended:

f_s = tensile unit stress in steel.

f_c = compressive unit stress in concrete.

E_s = modulus of elasticity of steel.

E_c = modulus of elasticity of concrete.

$n = \frac{E_s}{E_c}$.

M = moment of resistance, or bending moment in general.

A_s = steel area.

b = breadth of beam.
d = depth of beam to center of steel.
k = ratio of depth of neutral axis to depth d.
z = depth below top to resultant of the compressive stresses.
j = ratio of lever arm of resisting couple to depth d.
jd = $d-z$ = arm of resisting couple.
p = steel ratio $= \frac{A_s}{bd}$.

(*b*) *T-Beams.*

b = width of flange.
b' = width of stem.
t = thickness of flange.

(*c*) *Beams Reinforced for Compression.*

A' = area of compressive steel.
p' = steel ratio for compressive steel.
f_s' = compressive unit stress in steel.
C = total compressive stress in concrete.
C' = total compressive stress in steel.
d' = depth to center of compressive steel.
z = depth to resultant of C and C′.

(*d*) *Shear, Bond and Web Reinforcement.*

V = total shear.
V' = total shear producing stress in reinforcement.
v = shearing unit stress.
u = bond stress per unit area of bar.
o = circumference or perimeter of bar.
Σo = sum of the perimeters of all bars.
T = total stress in single reinforcing member.
s = horizontal spacing of reinforcing members.

(*e*) *Columns.*

A = total net area.
A_s = area of longitudinal steel.
A_c = area of concrete.
P = total safe load.

REPORT OF THE COMMITTEE ON REINFORCED-CONCRETE AND BUILDING LAWS.

At the meeting of the American Concrete Institute in Chicago in February, 1917, your committee presented a proposed Standard Building Regulations for the use of Reinforced Concrete. Since these regulations differed in several particulars from the recommendations of the Joint Committee on Concrete and Reinforced Concrete, it was decided by the convention to defer adoption of the American Concrete Institute Building Regulations until more data showing the reasons for the differences had been presented.

War conditions last year prevented further progress by the committee but this year the committee again took up its work. An analysis of the Joint Committee recommendations and the proposed regulations of the American Concrete Institute was made and similar sections of the two reports placed side by side for comparison. In several places the Joint Committee sections were considered equally as good or better than the American Concrete Institute, and in the revised regulations you will note that several of the changes have been to substitute the Joint Committee wording. After this analysis had been sent to members of the committee and replies from most of the members received, a meeting was held in New York City April 21 and 22, 1919, at which the recommendations of the different members were taken up and the revised regulations prepared for preprinting.

Several members of the committee have indicated that they would like to have further changes made in some sections before the regulations are finally adopted and it was expected that a majority of the committee would be present at a committee meeting this week, but two members have been unexpectedly prevented from attending. A majority of the committee was not present to agree to a report.

It was the opinion of the committee members present that the new sections in the revised regulations should be more thoroughly discussed at a meeting of a majority of the committee before being presented for adoption to the convention. Your committee is therefore prepared only to make a progress report.

The members of your committee present ask that the revised regulations as preprinted be accepted as a progress report and printed in the proceedings with such modifications as a majority of the committee decide should be made after considering the written discussion received and the discussion at this meeting.

E. J. Moore,
Chairman.

June 28, 1919.

[The regulations as revised by the committee, October, 1919, are printed beginning on the next succeeding page. None of the discussions of the members on the floor of the convention and none of the written discussions are printed in this Proceedings, because the report will be brought before the next annual convention for consideration. The discussion, in the nature of a minority report, of one member of the committee, is appended to the proposed regulations.—Editor.]

PROPOSED STANDARD BUILDING REGULATIONS FOR THE USE OF REINFORCED CONCRETE.

I. General.

1. The term "Reinforced Concrete," as used in these regulations, shall mean an approved mixture of portland cement with water and aggregates in which metal (generally steel) has been embedded in proportionately small sections, in such a manner that the metal and the concrete assist each other in taking stress. **Definition of Reinforced Concrete.**

2. Reinforced concrete may be used for all classes of buildings if the design is in accordance with good engineering practice, and stresses are calculated as indicated in these regulations. **Use.**

3. There shall be no limit upon the height of buildings of reinforced concrete, except as limited by general height restrictions, for all types of buildings or by the strength requirements in these regulations. **Height of Buildings.**

4. Before permission is granted by the Building Department to erect any reinforced-concrete building, complete general plans accompanied by specifications signed by the engineer or architect must be filed with the Building Department. This shall include a statement giving the dead- and live-loads, wind and impact, if any, and working stresses. Sufficient details shall be included in the plans submitted to make clear the exact dimensions and construction of the reinforced-concrete portions of the building and the arrangement of the reinforcement so as to permit computation of all stresses. Specifications shall state the qualities and proportions of the materials to be used. **Permits.**

Copies of approved plans and specifications must be left on file with the Building Department for public inspection until the building is completed.

5. Materials used for the concrete as well as for the reinforcement shall be carefully inspected and tested. The construction of the building shall be inspected in detail by a representative of the architect or engineer who will keep a complete record of the progress of the work, including dates of placing concrete and dates of removing forms. He shall also check the quantity of the materials used, and the placing of same in the different parts of the building. He shall insure that the work completed from day to day is kept moist for a period of not less than 5 days. These records shall be available for inspection by the Building Department. **Inspection.**

6. The Building Department may require the owner to make load tests on portions of the finished structure where there is a reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired, or where there is any doubt as to the sufficiency of the design. The test shall show that, with a load of twice the designed live load, the permanent deflection 7 days after load is removed to be not more than 20 per cent of the total deflection under the test load. Load tests shall not be made before the concrete has been in place 60 days. **Load Tests.**

In the event that the test proves unsatisfactory, the building may be tested again at a later time to determine when the concrete has developed adequate strength to permit design loading.

Posting of Floor. 7. The Building Department shall issue signed certificates to be posted on each floor of the building stating the allowable carrying capacity per square foot.

II. MATERIALS.

Specifications Cement. 8. Only portland cement shall be used in reinforced-concrete structures. Cement shall meet the requirements of the Standard Specifications for Cement of the American Society for Testing Materials as in effect at the time of the adoption of this regulation. (Standard 1, Am. Conc. Inst.)

Tests of Cement. 9. All cement used shall be tested and record of such tests shall be kept at the building site for inspection by the Building Department. No cement which has not met the requirements of the above specifications shall be used without the written approval of the Building Department.

Aggregates—General. 10. All aggregates shall be of clean material, free from dust, soft particles, lumps of clay, vegetable loam, and all organic matter.

Fine Aggregates. 11. Fine aggregate shall consist of sand, or the screenings of gravel or crushed stone, graded from fine to coarse, and passing, when dry, a screen having $\frac{1}{4}$ in. diameter holes, it preferably shall be of siliceous material, and not more than 30 per cent by weight shall pass a sieve having 50 meshes per linear inch.

Test of Fine Aggregates. 12. The suitable character of the cement, water, fine aggregate and coarse aggregate, mixed in the proportions to be used in the work and for the same length of mixing time, shall be established to the satisfaction of the Building Department by tests of 6 x 12 in. or 8 x 16 in. cylinders. Seven-day tests shall show at least 50 per cent and 28 day tests at least 100 per cent of the ultimate strength upon which the working stresses are based in accordance with Sections 40 and 41. Work may proceed upon the satisfactory completion of the 7-day tests. If the 28-day tests prove unsatisfactory, the water-cement ratio shall be sufficiently decreased or other materials substituted to provide the required ultimate strength. Additional cylinder tests may be required from time to time as the work progresses to further demonstrate the satisfactory strength of the concrete. Subsequent to the completion or preparation of such tests the materials used on the work shall not be changed without the consent of the Building Department.

Coarse Aggregates. 13. Coarse aggregates shall consist of crushed stone, gravel, or slag, which is retained on a screen having $\frac{1}{4}$ in. diameter holes and graded in size from small to large particles. The maximum size of the coarse aggregate shall be such that the concrete will flow freely around the reinforcement. Bank gravel shall be separated from the sand before mixing. Slag shall be clean, dense, air-cooled, blast furnace slag, weighing not less than 70 lb. per cu. ft. when loosely placed in the measure and containing not more than 1.3 per cent of sulphur as sulphides.

Cinders. 14. Cinders shall not be used as coarse aggregate in concrete for reinforced-concrete structures without tests acceptable to the Building Depart-

ment showing the strength of such concrete. Cinder concrete may be used for fireproofing, for floor and roof slabs, not exceeding 8 ft. span and for partitions. Where cinders are used as the coarse aggregate they shall be composed of hard, clean, vitreous clinker; free from sulphides, unburned coal or ashes.

15. The water used in mixing concrete shall be free from oil, acid, alkalies or organic matter. **Water.**

16. Steel for reinforcement of concrete shall conform to the requirements of the specifications of the American Society for Testing Materials for Concrete Reinforcement Bars, as in effect at the time of the adoption of this regulation. **Reinforcement.**

Cold-drawn steel wire made from billets may be used in floor and roof slabs, column hooping, and for temperature and shrinkage stresses. This steel shall have an elastic limit between fifty thousand (50,000) and sixty-five thousand (65,000) lb. per sq. in. and an ultimate strength of not less than eighty-five thousand (85,000) lb. per sq. in.

All reinforcing steel shall be free from flaking, rust, scale, or coatings of any character which will tend to reduce or destroy the bond.

III. DETAILS OF CONSTRUCTION.

17. Forms must be substantial and unyielding and sufficiently tight to prevent the leakage of mortar. Before placing concrete all forms shall be first thoroughly cleaned of all débris and preferably oiled to prevent adhesion of the concrete. **Forms.**

18. All bars must be carefully bent as required by plans. **Preparation of Reinforcement.**

19. Reinforcement shall be accurately located in the forms and secured against displacement. **Placing of Reinforcement.**

20. Where it is necessary to splice reinforcing steel, this shall be done, by providing a lap sufficient to transfer the stress between bars by bond and shear, or by a mechanical connection such as a screw coupling. Splices at point of maximum stress should be avoided. **Steel Splices.**

21. Vertical fill lines between two fills of concrete must be selected so that the resulting joint will have the least possible effect upon the strength of the structure. Before making the second fill, the concrete previously placed shall be thoroughly cleansed of foreign material and laitance, drenched and slushed with a mortar consisting of one (1) part portland cement and not more than two (2) parts fine aggregate. **Construction Joints.**

Construction joints for columns should be made at underside of floor construction, haunches and column capitals being considered as part of the floor construction, and should be poured monolithically. Where reinforced-concrete columns have flaring heads or where structural steel columns are used, concrete for slab and column heads may be poured immediately after the concrete for the column shaft.

In general, fill lines in floors should be selected near the center of spans of slabs, beams and girders. Unless a beam intersects a girder at this point, in which case the joint shall be offset a distance equal to twice the width of the beam. Where shear is present at the joint, adequate provision shall be made for resisting same by inclining joint or providing sufficient reinforcement.

Measuring Ingredients. 22. Methods of measuring of the various ingredients of concrete, including the water, shall be used which will secure separate and uniform measurements, of the proportions required. Measurements shall be made by volume; 94 lb. of cement to be considered as a cubic foot.

Mixing—General. 23. The ingredients of concrete shall be thoroughly mixed to the desired consistency and the mixing shall continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous.

Machine Mixing. 24. In mixing by machine a mixer of a type which insures the uniform distribution of the materials throughout the mass, and in which the required portions of the water-cement and aggregates can be accurately measured, shall be used.

Hand Mixing. 25. When it is necessary to mix by hand, the mixing shall be done on a watertight platform, and all ingredients shall be turned together at least six times and until the resulting mass is homogeneous in appearance and color.

Consistency. 26. The materials must be mixed wet enough to produce a concrete of such a consistency that it will flow sluggishly into the forms and about the metal reinforcement, and at the same time can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

Re-tempering. 27. Mortar or concrete shall not be re-mixed with water and used after it has partly set.

Placing of Concrete. 28. Concrete after the completion of the mixing shall be transported as rapidly as practicable from the place of mixing to the place of final deposit. The concrete shall be deposited in such a manner that it will flow sluggishly around the steel reinforcement and shall be rammed or agitated by suitable tools in such a manner as to produce thoroughly compact concrete.

Where concrete is conveyed by spouting, the plant shall be of such a size and design as to ensure a practically continuous stream in the spout. The angle of the spout with the horizontal shall be such as to allow the concrete to flow without a separation of the ingredients; in general an angle of about 27°, or one vertical to two horizontal will be required.

Placing in Water. 29. Concrete shall not be placed in water unless unavoidable; if necessary to do this a tremie or other method demonstrated to be especially effective shall be used to prevent the cement from being separated from the aggregate.

Finishing. 30. The concrete at the end of each fill shall be cleaned of laitance or other deleterious material which would detract from the quality of the concrete. After forms are removed, any porous sections of concrete shall be cleaned out and filled in a manner to meet the approval of the Building Department.

Protection in Warm Weather. 31. Newly placed concrete shall be protected from rapid drying and kept damp for a period of at least 5 days.

Protection in Cold Weather. 32. Concrete shall not be mixed or deposited during freezing temperatures, unless it is maintained at a temperature not less than 50° F. during mixing, placing, and for at least 72 hours thereafter, and until the concrete is thoroughly hardened.

Removal of Forms. 33. Under no consideration shall forms be removed until the concrete has hardened sufficiently to permit their removal with safety.

Where there is danger of frozen concrete being mistaken for properly hardened concrete, heat shall be applied before tests for hardness are made.

34. Before a section of form is removed, shoring shall be provided as necessary to carry the weight of the new concrete and other loads brought upon the construction in acting as a support for upper floors. Careful consideration must be given to the loads carried and the strength of the new concrete before any shoring is removed. **Temporary Supports.**

IV. DESIGN.

35. All reinforced-concrete construction shall be designed to meet the conditions of loading (including bending in columns) without stressing the materials used beyond the safe working stresses specified. **Conditions.**

36. The dead-loads shall be the weight of the permanent structure. The weight of reinforced stone, gravel or slag concrete shall be taken as 144 lb. per cu. ft.; the weight of cinder concrete as 100 lb. per cu. ft **Dead Loads.**

37. The live-load shall be the working or variable load for which the structure is designed. **Live Loads.**

38. All parts of a structure shall be designed to carry safely the entire combined dead- and live-loads with the exception that the loads on columns and foundations may be reduced by considering that columns in top story carry the total live- and dead-load above them; columns in next to top story carry the total dead-load and eighty-five (85) per cent of the total live-load above; columns in the next lower story, the total dead-load and eighty (80) per cent of the total live-load above; and thus on downward, reducing at each story the percentage of total live-loads carried, by 5, until a reduction of fifty (50) per cent is reached. The columns in this and in every story below this point, shall be proportioned to carry the total dead load and at least fifty (50) per cent of the total live load of all the floors and roofs above them. **Reduction of Loads.**

For warehouses the increment of reduction per story shall be $2\frac{1}{2}$ per cent. instead of 5 per cent.

39. As a basis for calculations for the strength of reinforced-concrete construction the following assumptions shall be made: **General Assumptions.**

(*a*) Calculations shall be made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.

(*b*) A plane section before bending remains plane after bending.

(*c*) The modulus elasticity of concrete in compression within the usual limits of working stresses is constant.

(*d*) In calculating the moment of resistance of beams, the tensile stresses in the concrete are neglected.

(*e*) Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses the two materials will, therefore, be stressed in proportion to their moduli of elasticity.

(*f*) The ratio of the modulus of elasticity of concrete shall be taken as follows:

1. One-fortieth that of steel when the strength of the concrete is taken as not more than eight hundred (800) lb. per sq. in.

2. One-fifteenth that of steel when the strength of the concrete is taken as greater than twelve hundred (1200) lb. per sq. in. or less than twenty-two hundred (2200) lb. per sq. in.
3. One-twelfth that of steel when the strength of the concrete is taken as greater than twenty-two hundred (2200) lb. per sq. in. or less than thirty three hundred (3300) lb. per sq. in.
4. One-tenth that of of steel when the strength of the concrete is taken as greater than thirty-three (3300) lb. per sq. in.

Strength of Materials.

40. The ultimate strength of concrete shall be that developed at an age of 28 days in cylinders 8 in. in diameter and 16 in. in length or 6 in. in diameter and 12 in. in length of the consistency and proportions to be used in the work, made and stored under laboratory conditions, but in no case shall the values exceed those allowed for granite in the table below. In the absence of definite knowledge in advance of construction as to just what strength may be developed, the following values may be used:

TABLE OF STRENGTHS OF DIFFERENT MIXTURES OF CONCRETE.

(In Pounds per Square Inch.)

Aggregate	1:3	1:4½	1:6	1:7½	1:9
Granite, trap rock	3300	2800	2200	1800	1400
Gravel, hard limestone, hard sandstone and approved slag	3000	2500	2000	1600	1300
Soft limestone and sandstone	2200	1800	1500	1200	1000
Cinders	800	700	600	500	400

Safe Working Stresses.

41. Reinforced-concrete structures shall be so designed that the stresses, figured in accordance with these regulations, in pounds per square inch, shall not exceed the following:

(*a*) Extreme fiber stress in concrete in compression 37½ per cent of the compressive strength specified in Section 40. Adjacent to the support of continuous members, 41 per cent provided the member frames into a mass of concrete projecting at least 50 per cent of the least dimension of the member on all sides of the compression area of the member.

(*b*) Concrete in direct compression 25 per cent of the compressive strength specified in Section 40.

(*c*) Shearing stress in concrete when main steel is not bent and when steel is not provided to resist diagonal tension—2 per cent of the compressive strength specified in Section 40.

(*d*) Where punching shear occurs, provided the diagonal tension requirements are met, a shearing stress of 7½ per cent of the compressive strength will be allowed.

(*e*) Vertical shearing stress, where the member is properly reinforced to resist diagonal tension, 7½ per cent of the ultimate compressive strength of the concrete as specified in Section 40, provided that in freely supported or partially restrained members the longitudinal reinforcing bars

shall be carried over the support and anchored against slipping by means full half turns (180° hooks) of a diameter not less than seven times the diameter of the bars, or by some fully equivalent method of anchorage, and that in continuous or fully restrained members the longitudinal reinforcement in the bottom of the member shall be anchored as above or shall be carried beyond the point of inflection into the compression area of the member for such a distance that the computed total bond resistance (at the values permitted in Section 41 (*f*) or 41 (*g*)) on the portion of the bar between the point of inflection and the end of the bar shall equal the maximum tension permitted in the bar by Section 41(*i*).

(*f*) Bond stress between concrete and plain reinforcing bars—4 per cent. of the compressive strength.

(*g*) Bond stress between concrete and approved deformed bars—5 per cent of the compressive strength.

(*h*) Compression applied to a surface of concrete of at least twice the loaded area, a stress of 50 per cent of the compressive strength shall be allowed over the area actually under load.

(*i*) Tensile stress in steel—16,000 lb. per sq. in., except that for steel having an elastic limit of at least 50,000 lb., a working stress of 18,000 lb. per sq. in. will be allowed.

Girder, Beam, and Slab Construction

42. In determining the bending moment in slabs, beams and girders, the load carried by the member shall include both the dead- and the live-loads.

The span of the member shall be the distance center to center of supports but not to exceed the clear span plus the depth of the member, except that for continuous or fixed members framing into other reinforced concrete members the clear span may be used.

For continuous members supported upon brackets making an angle of not more than 45° with the vertical, and having a width not less than the width of the member supported, the span shall be the clear distance between brackets plus one-half the total depth of the member.

If the brackets make a greater angle than 45° with the vertical, only that portion of the bracket within the 45° slope shall be considered. Maximum negative moments are to be considered as existing at the end of the span as here defined.

For members uniformly loaded the bending moment shall be assumed as $\frac{WL}{F}$, where W = total load; L = span; and F = 8 for members simply supported, 10 for both negative and positive bending moment for members restrained at one end and simply supported or partially restrained at the other, and 12 for both negative and positive bending moment for members fixed or continuous at both supports. The above bending moments for continous members apply only when adjacent spans are approximately equal.

A special condition of loading to be reduced to equivalent uniformly distributed loading in accordance with approved engineering practice. For members having one end simply supported or partially restrained, at least fifty (50) per cent of the tension reinforcement required at center of span

shall be bent up and adequately anchored to take bending moment at exterior support.

At the ends of continuous beams, the amount of negative moment which will be developed in the beam will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. In the ordinary cases a moment of $\frac{wl^2}{16}$ may be taken; for small beams running into heavy columns this should be increased but not to exceed $\frac{wl^2}{12}$.

Slabs. 43. The main tensile reinforcement shall not be farther apart than two times the thickness of the slab. For slabs designed to span one way, steel having an area of at least two-tenths of one per cent (0.2%) of section of slab shall be provided transverse to main reinforcement, and this transverse reinforcement shall be further increased in the top of the slab over girders to prevent cracking and the main steel in slabs parallel and adjacent to girders may be reduced accordingly. Where openings are left through slabs, extra reinforcement shall be provided to prevent local cracks developing. This reinforcement shall in no case be less than ¼ sq. in. in section and must be securely anchored at ends. Floor finish when placed monolithic may be considered part of the structural section.

T-Beams and Girders. 44. Where adequate bond and shearing resistance between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall not exceed on either side of the beam one-sixth of the span length of the beam nor be greater than six times the thickness of the slab on either side of the beam, nor greater than one-half of the distance between beams on either side, the measurements being taken from edge of web.

Web reinforcement for concrete beams and girders shall be designed by the formula (subject to the limitations of section 41 (*e*)).

$$A_s = \frac{2}{3}\,\frac{Vs}{f_s jd} \quad \text{(also written } v = 1.5\ rf_s\text{) in which}$$

A_s = area of one web bar or stirrup.
s = spacing of bars normal to their direction.
f = allowable stress in web and steel as in section 41 (*i*).
jd = effective depth of beam—between centroids of tensile and compressive stresses.
V = total shear at section.
v = unit vertical shearing stress at section.
r = ratio of web steel to web concrete, or the percentage computed in a plane at right angles to its direction.

Web reinforcement shall extend to and be anchored about the horizontal reinforcement in both the top and bottom of the beam. Bent-up longitudinal steel may be considered a part of the web reinforcement provided it is adequately anchored against slipping by hooked ends, or by extension into the adjoining span. The diameter of detached web reinforcing bars shall not

exceed $\frac{1}{50}$ of their length between points of anchorage to the longitudinal steel. Web members shall be spaced not to exceed three-fourths of the effective depth of the beam, and not to exceed twice the thickness of the web in that portion where the web stresses exceed the allowable value of unreinforced concrete in shear. Web members may be placed vertically (at right angles to the axis) or at an angle not less than 30° to the axis of the beam (inclined to take tension) or at any intermediate angle between these limits.

Tile and Joist Floors.

45. Wherever floors are built with a combination of tile or other fillers between reinforced-concrete joists, the following rules regarding the dimensions and methods of calculations of construction shall be observed:

(*a*) Wherever a portion of the slab above the fillers is considered as acting as a T-beam section, the slab portion must be cast monolithic with the joist and have a minimum thickness of two (2) in.

(*b*) Wherever porous fillers are used which will absorb water from the concrete, care must be taken to thoroughly saturate same before concrete is placed.

(*c*) All regulations given above for beam and girder floors shall apply to tile and joist floors.

(*d*) The sections of fillers shall be together and all joints reasonably tight before concrete is placed.

Flat-Slab or Girdless Floors.

46. Continuous flat-slab floors, reinforced with steel rods or mesh and supported on spaced columns in orderly arrangement, shall conform to the following requirements:

(*a*) *Notation and Nomenclature.*—In the formulas let

w = total dead- and live-load in pounds per square foot of floors.

l_1 = span in feet center to center of columns parallel to sections on which moments are considered.

l_2 = span in feet center to center of columns perpendicular to sections on which moments are considered.

c = average diameter of column capital in feet at plane where its thickness is $1\frac{1}{2}$ in.

q = distance from center line of the capital to the center of gravity of the periphery of the half capital divided by $\frac{1}{2}c$. For round capitals q may be considered as two-thirds and for square capitals as three-quarters.

t = total slab thickness in inches.

L = average span in feet center to center of columns, but not less than 0.9 of the greater span.

The column head section, mid section, outer section, and inner section are located and dimensioned as shown in Fig. 1. Corresponding moments shall be figured on similar sections at right angles to those shown in Fig. 1.

(*b*) *Structural Variations.*—Flat-slab floors may be built with or without caps, drops or paneled ceilings. These terms are illustrated in Fig. 2.

Flat Slabs (cont'd).

Where caps are employed they shall be considered a part of the columns and the column capital dimension c shall be found by extending the lines of the capital to an intersection with the plane of the under surface of the slab as indicated in Fig. 2*b*. The cap shall be large enough to enclose this extension of the capital lines.

The column capital profile shall not fall at any point inside an inverted cone drawn, as shown in Fig. 2*a*, from the periphery of the designed capital of diameter c and with a base angle of 45°. The diameter of the designed capital c shall be taken where the vertical thickness of the column capital is at least $1\frac{1}{2}$ in.

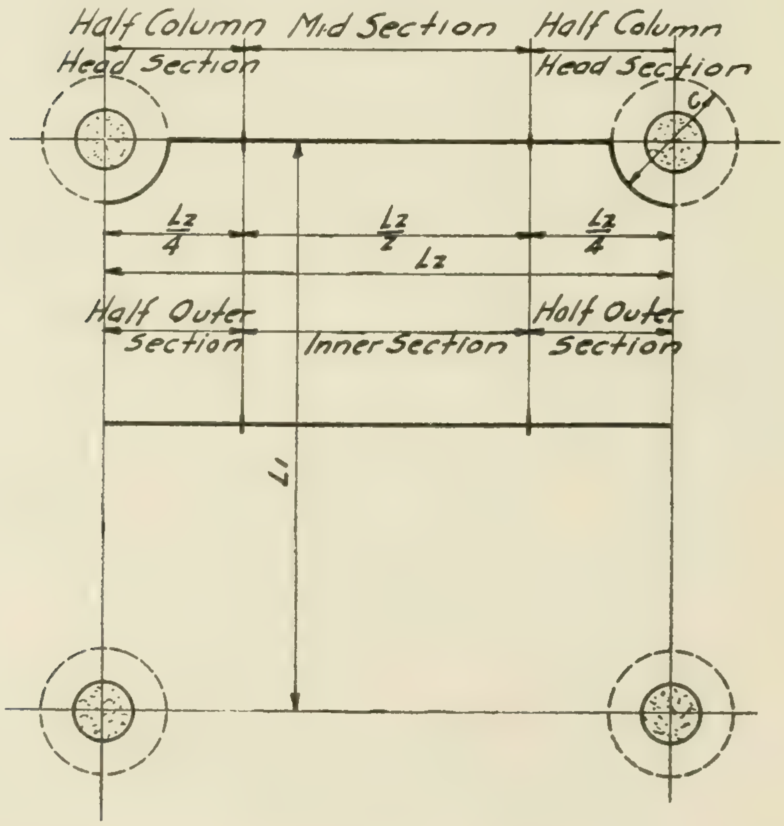


FIG. 1.

The drop, where used, shall not be less than 0.3 L in width.

Where paneled ceilings are used the paneling shall not exeecd one-third of the slab thickness in depth and the dimension of the paneling shall not exceed 0.6 of the panel dimension. (See Fig. 2*c*.)

(*c*) *Slab Thickness.*—The slab thickness shall not be less than $t = 0.02\,L\sqrt{w} + 1$ *in.*

In no case shall the slab thickness be less than $\frac{1}{32}L$ for floor slabs nor less than $\frac{1}{40}L$ for roof slabs

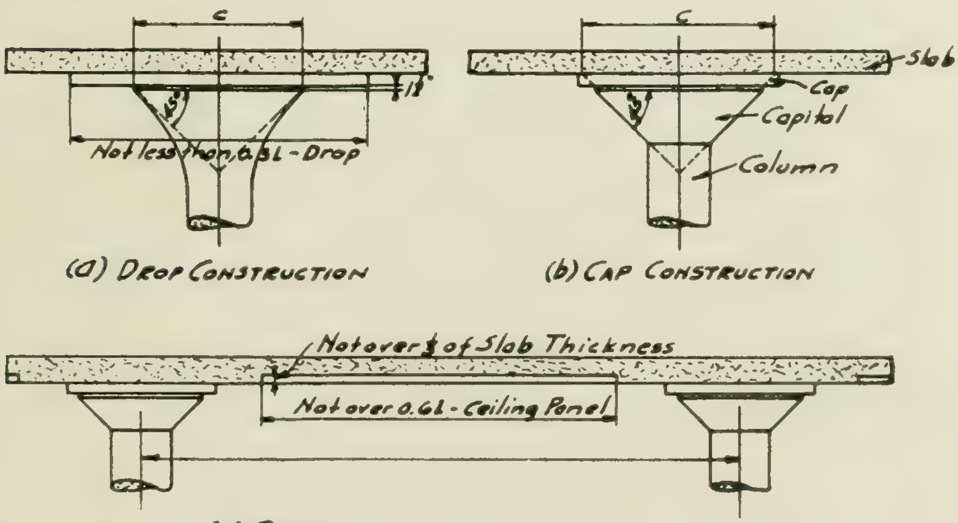
(*d*) *Design Moments.*—The numerical sum of the positive and **Flat Slabs (cont'd).** negative moments in foot pounds shall not be less than 0.09 $wl_1\ (l_2-qc)^2$. Of this total amount not less than 40 per cent shall be resisted in the column head sections. Where a drop is used, not less than 50 per cent shall be resisted in the column head sections.

Of the total amount not less than 10 per cent shall be resisted in the mid section.

Of the total amount not less than 18 per cent shall be resisted in the outer sections.

Of the total amount not less than 12 per cent shall be resisted on the inner sections.

The balance of the moment shall be distributed between the various sections as required by the physical details and dimensions of the particular design employed.



(a) Drop Construction (b) Cap Construction

(c) Panelled Ceiling Construction

Fig. 2

(*e*) *Exterior panels.*—The negative moments at the first interior row of columns and the positive moments at the center of the exterior panel on sections parallel to the wall, shall be increased 20 per cent over those specified above for interior panels. If girders are not provided along the column line, the reinforcement parallel to the wall for negative moment in the column head section and for positive moment in the outer section adjacent to the wall, shall be altered in accordance with the change in the value of c. The negative moment on sections at the wall and parallel thereto should be determined by the conditions of restraint, but must never be taken less than 80 per cent of those for the interior panels.

(*f*) *Reinforcement.*—In the calculation of moments all the reinforcing bars which cross the section under consideration and which fulfil the requirements given under "Arrangement of Reinforcement" may be used. For a column head section reinforcing bars parallel to the straight

Flat Slabs (cont'd).

portion of the section do not contribute to the negative resisting moment for the column head section in question. The sectional area of bars, crossing the section at an angle, multiplied by the sine of the angle between these bars and the straight portion of the section under consideration may be taken to act as reinforcement in a rectangular direction.

(g) *Point of Inflection.*—For the purpose of making calculations of moment at sections away from the sections of negative moment and positive moment already specified, the point of inflection shall be taken at a distance from center line of columns equal to $\frac{1}{5}$ $(l_2 - qc) + \frac{1}{2}$ qc. This becomes $\frac{1}{5}$ $(l_2 + c)$ where capital is circular. For slabs having drop panels the coefficient of $\frac{1}{4}$ should be used instead of $\frac{1}{5}$.

(h) *Arrangement of Reinforcement.*—The design should include adequate provision for securing the reinforcement in place so as to take not only the maximum moments but the moments of intermediate sections. If bars are extended beyond the column capital and are used to to take the bending moment on the opposite side of the column, they must extend to the point of inflection. Bars in diagonal bands used as reinforcement for negative moment should extend on each side of the line drawn through the column center at right angles to the direction of the band a distance equal to 0.35 of the panel length, and bars in the diagonal bands used as reinforcement for positive moment, should extend on each side of the diagonal through the center of the panel a distance equal to 0.35 of the panel length. Bars spliced by lapping and counted as only one bar in tension shall be lapped not less than 80 diameters if splice is made at point of maximum stress and not more than 50 per cent of the rods shall be so spliced at any point in any single band or in any single region of tensile stress. Continuous bars should not all be bent up at the same point of their length, but the zone in which this bending occurs should extend on each side of the assumed point of inflection.

(i) *Tensile and Compressive Stresses.*—The usual method of calculating the tensile and compressive stresses in the concrete and in the reinforcement, based on the assumptions for internal stresses, should be followed. In the case of the drop panel, the section of the slab and drop panel may be considered to act integrally for a width equal to the width of the column head section. Within the column head section the allowable compression may be increased as prescribed in Section 41 for continuous members.

(j) *Provision for Diagonal Tension and Shear.*—In calculations for the shearing stress which is to be used as the means of measuring the resistance to diagonal tension stress, it shall be assumed that the total vertical shear on a column head section constituting a width equal to one-half the lateral dimension of the panel, for use in determining critical shearing stresses, shall be considered to be one-fourth of the total dead- and live-load on a panel for a slab of uniform thickness, and to be 0.3 of the sum of the dead- and live-loads on a panel for a slab with drop panels. The formula for shearing unit stress shall be $v = \frac{0.25W}{bjd}$ for slabs of uni-

form thickness and $v = \frac{0.30W}{bjd}$ for slabs with drop panels, where W is the sum of the dead- and live load on a panel, b is half the lateral dimension of the panel measured from center to center of columns, and jd is the lever arm of the resisting couple at the section. **Flat Slabs (cont'd).**

The calculation for punching shear shall be made on the assumption of a uniform distribution over the section of the slab around the periphery of the column capital and also of a uniform distribution over the section of the slab around the periphery of the drop panel, using in each case an amount of vertical shear greater by 25 per cent than the total vertical shear on the section under consideration.

The values of working stresses should be those recommended for diagonal tension and shear in Section 41.

(k) *Walls and Openings.*—Additional slab thickness, girders, or beams shall be provided to carry walls and other concentrated loads which are in excess of the working capacity of the slab. Beams should also be provided in case openings in the floor reduce the working strength of the slab below the required carrying capacity.

(l) *Unusual Panels.*—The coefficients, steel distribution, and thicknesses recommended are for slabs which have three or more rows of panels in each direction and in which the sizes of the panels are approximately the same. For structures having a width of one or two panels and also for slabs having panels of markedly different sizes, an analysis should be made of the moments developed in both slab and columns and the values given herein modified accordingly.

(m) *Oblong Panels.*—The requirements of design herein given for flat slab floors do not apply for oblong panels where the long side is more than four-thirds of the short side.

(n) *Bending Moments in Columns.*—Provision shall be made in both wall columns and interior columns for the bending moment which will be developed by unequally loaded panels, eccentric loading, or uneven spacing of columns. The amount of moment to be taken by a column will depend on the relative stiffness of columns and slab, and computations may be made by rational methods such as the principle of least work or of slope and deflection. Generally the largest part of the unequalized negative moment will be transmitted to the columns and the columns shall be designed to resist this bending moment. Especial attention shall be given to wall columns and corner columns. Column capitals shall be designed, and reinforced where necessary, with these conditions in mind.

The resistance of any column to bending in a direction parallel to l_2 shall be not less than $0.022\ w_1 l_1\ (l_2 - qc)^2$, in which w_1 is the designed live load per square foot. In determining the resistance to be provided in exterior columns in a direction perpendicular to the wall the full dead and live load w shall be used in the above formula in place of w_1. The moment in such exterior columns may be reduced by the balancing moment of the weight of the structure which projects beyond the supporting wall column center line.

Flat Slabs (cont'd).

Where the column extends through the story above, the resisting moment shall be divided between the upper and the lower columns in proportion to their stiffness. Calculated combined stresses due to bending and direct load shall not exceed by more than 50 per cent the stresses allowed for direct load.

Columns—General.

47. Reinforced-concrete columns, for the working stresses hereinafter specified, shall have a gross width or diameter not less than one-fifteenth of the unsupported height nor less than twelve (12) in. All vertical reinforcement shall be secured against lateral displacement by steel ties not less than $\frac{1}{4}$ in. in diameter, placed not farther apart than 15 diameters of the vertical rods or more than 12 in.

For columns supporting flat slab floors or roofs, the diameter shall be not less than one-thirteenth of the distance between columns.

The length of columns should be taken as the maximum unstayed length.

Columns with Longitudinal Reinforcement.

48. For columns having not less than 0.5 per cent nor more than 4 per cent of vertical reinforcement, the allowable working unit stress for the net section of the concrete shall be 25 per cent of the compressive strength specified in Section 40, and the working unit stress for the steel shall be based upon the ratio of the moduli of elasticity of the concrete and steel. Concrete to a depth of $1\frac{1}{2}$ in. shall be considered as protective covering and not a part of the net section.

Columns with Longitudinal and Lateral Reinforcement.

49. Columns, having not less than 1 per cent nor more than 4 per cent of vertical reinforcement and not less than 0.5 per cent nor more than 2 per cent of lateral reinforcement in the form of hoops or spirals spaced not farther apart than one-sixth of the outside diameter of the hoops or spirals nor more than 3 in.; shall have an allowable working unit stress for the concrete within the outside diameter of the hoops or spirals equal to 25 per cent of the compressive strength of the concrete, as given in Section 40, and a working unit stress on the vertical reinforcement equal to the working value of the concrete multiplied by the ratio of the specified moduli of elasticity of the steel and concrete, and a working load for the hoops or spirals determined by considering the steel in hoops or spirals as five times as effective as longitudinal reinforcing steel of equal volume. The percentage of lateral reinforcement shall be taken as the volume of the hoops or spirals divided by the volume of the enclosed concrete in a unit length of column. The hoops or spirals shall be rigidly secured at each intersection to at least four (4) verticals to insure uniform spacing. The percentage of longitudinal reinforcement used shall be not less than the percentage of the lateral reinforcement.

50. For steel columns filled with concrete and encased in a shell of concrete at least 3 in. thick where the steel is calculated to carry the entire load, the allowable stress per square inch shall be determined by the following formula; $18000 - 70\frac{L}{R}$ but shall not exceed 16,000 lb.—where L = unsupported length in inches and R = least radius of gyration of steel section in inches. The concrete shell shall be reinforced with wire mesh or hoops weighing at least 0.2 lb per sq. ft.

When the details of the structural steel are such as to fully enclose or encase the concrete, or where a spiral of not less than one-half of 1 per cent of the core area and with a pitch of not more than three inches is provided for this purpose, the concrete inside the column core or spiral may be loaded to not more than 25 per cent of the ultimate strength specified in Section 40, in addition to the load on the steel column figured as above.

Composite columns having a cast iron core or center surrounded by concrete which is enclosed in a spiral of not less than one-half of 1 per cent of the core area and with a pitch of not more than three inches, may be figured for a stress of $12000 - 60\,L/R$, but not over 10,000 lb. per sq. in. on the cast iron section and of not more than 25 per cent of the compressive strength specified in Section 40 on the concrete within the spiral or core. The diameter of the cast iron core shall not exceed one-half of the diameter of the spiral.

Footings—General. 51. Symmetrical, concentric column footings shall be designed for punching shear, diagonal tension and bending moment.

Punching Shear in Footings. 52. The area effective to resist punching shear in column footings shall be considered as the area having a width equal to the perimeter of the column or pier and a depth equal to the depth of footing from top to center of reinforcing steel.

Diagonal Tension in Footings. 53. Shearing stresses as indicative of diagonal tension shall be measured in footings on vertical sections distant from the face of the pier or column equal to the depth of the footing from top to center of reinforcing steel.

Bending Moment in Footings. 54. The bending moment in isolated column footings at a section taken at edge of pier or column shall be determined by multiplying the load on the quarter footing (after deducting the quarter pier or column area) by six-tenths of the distance from the edge of pier or column to the edge of footing. The effective area of concrete and steel to resist bending moment shall be considered as that within a width extending both sides of pier or column, a distance equal to depth of footing plus one-half the remaining distance to edge of footing, except that reinforcing steel crossing the section other than at right angles, shall be considered to have an effective area determined by multiplying the sectional area by the sine of the angle between the bar and the plane of the section.

Bond Stresses n Footings. 55. In designing footings, careful consideration must be given to the bond stresses which will occur between the reinforcing steel and the concrete.

Walls—General. 56. Walls shall be reinforced by steel rods running horizontally and vertically. Walls having an unsupported height not exceeding fifteen times the thickness may be figured the same as columns. Walls having an unsupported height not more than twenty-five times the thickness may be figured to carry safely a working stress of 12½ per cent of the compressive strength specified in Section 40.

Exterior Walls. 57. Exterior walls shall be designed to withstand wind loads or loads from backfill. The thickness of wall shall in no case be less than 4 in.

Protection. 58. The reinforcement in columns and girders shall be protected by a minimum thickness of 2 in. of concrete; in beams and walls by a minimum of 1½ in.; in floor slabs by a minimum of ¾ in.; in footings by a minimum of 3 in.

MINORITY COMMITTEE REPORT ON PROPOSED STANDARD BUILDING REGULATIONS FOR THE USE OF REINFORCED CONCRETE.

[The following paragraphs are submitted by Mr. Edward Godfrey, member of the Committee on Reinforced-Concrete Building Laws, as a dissenting opinion to the "Proposed Standard Building Regulations for the Use of Reinforced Concrete," prepared as a preliminary document by the committee and sent out to the members in May, 1919.—EDITOR.]

Mr. Edward Godfrey dissents from the report in the whole matter of stirrups and their treatment. He would give stirrups and short shear members no recognition, for the reason that he holds that they have not shown themselves to have any definite value in tests and that analysis fails to show that any definite value can be ascribed to them; he also believes that dependence on stirrups to take end shear has resulted in much unsafe construction and some failures. He would take care of diagonal tension by bending up some of the main reinforcing rods and anchoring them for their full tensile strength beyond the edge of support. He recommends that bends be made close to the supports for the upper bends and at quarter points for the lower bends in beams carrying uniform load. For girders carrying beams bends should be made under the beams. For anchorage he recommends that the rod should extend 40 to 50 diameters beyond the point where it intersects a line drawn at 45° with the horizontal from the bottom of the beam at the face of the support.

He recommends that the stress in bent up rods be assumed to be that obtained by multiplying the excess of shear over that taken by the concrete (at 40 to 50 lb. per sq. in.) by the secant of the inclination of the rod with the vertical.

Mr. Godfrey also dissents from all parts of the report relating to rodded columns or columns having longitudinal rods without close-spaced hooping, for the reason that he holds that such reinforcement has not shown itself to have any definite value in tests on columns, and that analysis fails to show that any definite value can be ascribed to it, when such analysis takes into account the necessity for toughness in all columns; he also believes that dependence on such reinforcement has led to much unsafe construction and many failures. He would recognize as reinforced-concrete columns only such columns as have in addition to the longitudinal rods a complete system of close-spaced hooping. He recommends the standardization of hooped columns and suggests that columns be reinforced by a coil or hoops of round steel having a diameter one-fortieth of that of the external diameter of the column and eight upright rods wired to the same, the pitch of the coil being one-eighth of the column diameter. He would consider available for resisting compressive stress, the entire area of the concrete of a circular column or of an octagonal column, but no part of the longitudinal rods or hooping.

In square columns only 83 per cent of the area of concrete would be considered available. The compression he would recommend in columns (for 2000 lb. concrete) would be

$$P = 670 - 12\ l/d$$

where P = allowed compression in lb. per sq. in.
l = length of column in inches
d = diameter of column in inches

Mr. Godfrey dissents from the parts of the report that allow lower unit stresses than those allowed by the Joint Committee and that allow lower moment coefficients for flat slabs.

REPORT OF THE COMMITTEE ON CONCRETE ROADS AND PAVEMENTS.

The committee has thought that this year was hardly the time to revise the specifications on concrete roads; so many new practices are being evolved that it is thought better to bring out a specification next year that will not have to be changed. Last year we brought before you a "Recommended Practice" that we asked leave to have printed in the Proceedings of the Society, with the idea that it would be taken up this year by letter ballot. This year it was sent to the members of the committee and they were asked to give suggestions with regard to its amendment. Those suggestions have been formulated in the amendments presented herewith.

H. E. BREED, *Chairman.*

[The amendments to the "Recommended Practice for Concrete Road and Street Construction," Proc. Am. Conc. Inst., Vol. XIV, 1918, p. 518, were read at the convention of the Institute and discussed on the floor. A transcript of this discussion was submitted to the committee, as a result of which the amendments were revised, as printed on p. 405 of this volume. By order of the convention, these amendments, with their reference to the previously printed "Recommended Practice," are to be printed in this volume of "Proceedings" and are to be brought up for action at the next convention of the Institute.—EDITOR.]

AMENDMENTS TO RECOMMENDED PRACTICE FOR CONCRETE ROADS AND STREET CONSTRUCTION.

(The section numbers refer to the "Recommended Practice," as printed in the Proceedings, Am. Conc. Inst., Vol. XIV, 1918, p. 518, *et seq.*)

The cuts shown in the report are revisions of drawings of the same figure in the Proceedings, Am. Conc. Inst., Vol. XIV, 1918, p. 518, *et seq.* The cuts shown in the report, are revisions of drawings of the same figure numbers in the previous "Recommended Practice.")

Section 4.—(To be amended as follows. To follow directly after the first paragraph of this section.)

Where screenings are available and sand is of higher cost a mixture of screenings and sand, meeting the test, may be used. Fine sand can ofttimes be made suitable by the addition of screenings.

Section 15.—(Cross out the second paragraph of this section and substitute the following.)

Unless otherwise indicated on the drawings, all curves shall be superelevated as follows:

Up to 250-ft. radius, ¾ in. per ft.; 250 to 800-ft. radius, ½ in. per ft. On curves with a greater radius than 800 ft., the usual cross-section on level stretches shall be followed around the curves.

Unless otherwise indicated on the drawings, the grade of the center line is to be maintained and superelevation secured by raising the outside and depressing the inside edges of the pavement. In changing from the straight to the curved section, when the full superelevation is reached, the surface shall be without crown or dish.

Section 18.—(Cut out the tenth paragraph and insert in its place the following.)

A ½-in. joint in the curb shall be made at every joint in the pavement wherever it is provided that joints shall be constructed in the pavement. Joints shall extend entirely through the pavement and the curb and shall be in perfect alignment both transversely and vertically with the joint material.

Section 26 A.—(To follow last paragraph of Section 26.)

Mixer Loader.

Machines to load the hopper are in use and it is recommended that by their use a saving in labor can be effected. Their use also insures capacity operation for large sized mixers.

Section 27 A.—(To follow Section 27.)

Consistency.

The materials shall be mixed to such consistency that a 6 x 12 in. cylindrical form filled with the concrete as mixed shall have a slump upon

removing the cylindrical form not to exceed 5 in., unless the surface is to be machine finished, in which case the consistency shall be such that the slump, upon removing the cylindrical form shall not exceed 2 in.

Section 29.—(Add to paragraph as follows.)

All mortar and earth shall be removed from the forms, and the side in contact with the concrete shall be given a wash of oil or soap before they are used.

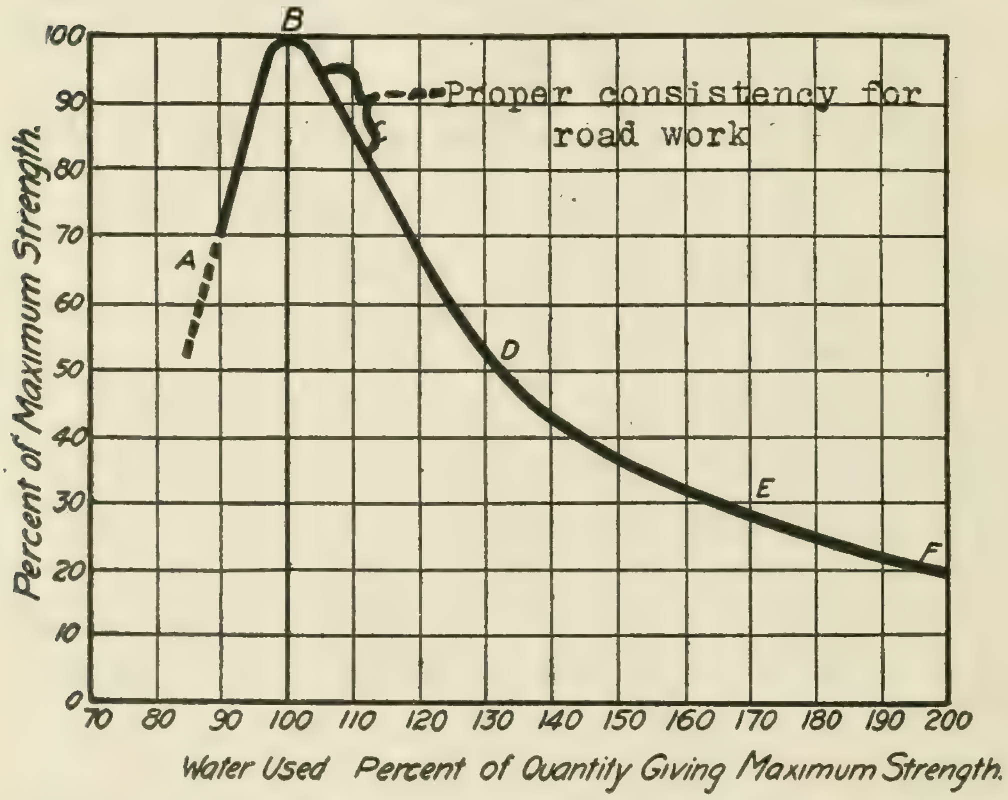


FIG. 15.—CURVE SHOWING EFFECT OF WATER USED IN MIXING ON STRENGTH OF CONCRETE.

Point B represents maximum strength obtainable from mix. The bracket indicates the consistency which should be arrived at in road work. From D to F strength decreases rapidly as water is increased.

Section 30.—(The graph in this section should be corrected as shown in the new Fig. 15.)

Section 30.—Third paragraph. (Cut out all after the third sentence.)

Section 36.—(After the third paragraph add as follows.)

Where a finishing machine is used the joint filler should not protrude above the surface.

(Fig. 26 to be corrected as shown.)

Section 37.—(Make the first paragraph read as follows:.

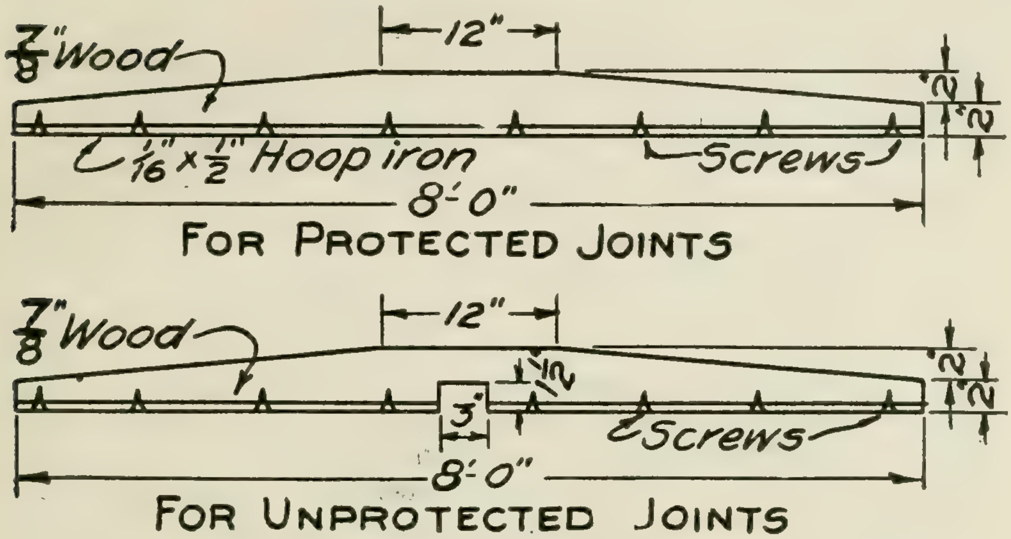


FIG. 26.—STRAIGHT-EDGE FOR SECURING SMOOTH RIDING JOINTS.

(Steel protection plates for joints are not recommended, but if used should be tested for height with straight-edge shown.)

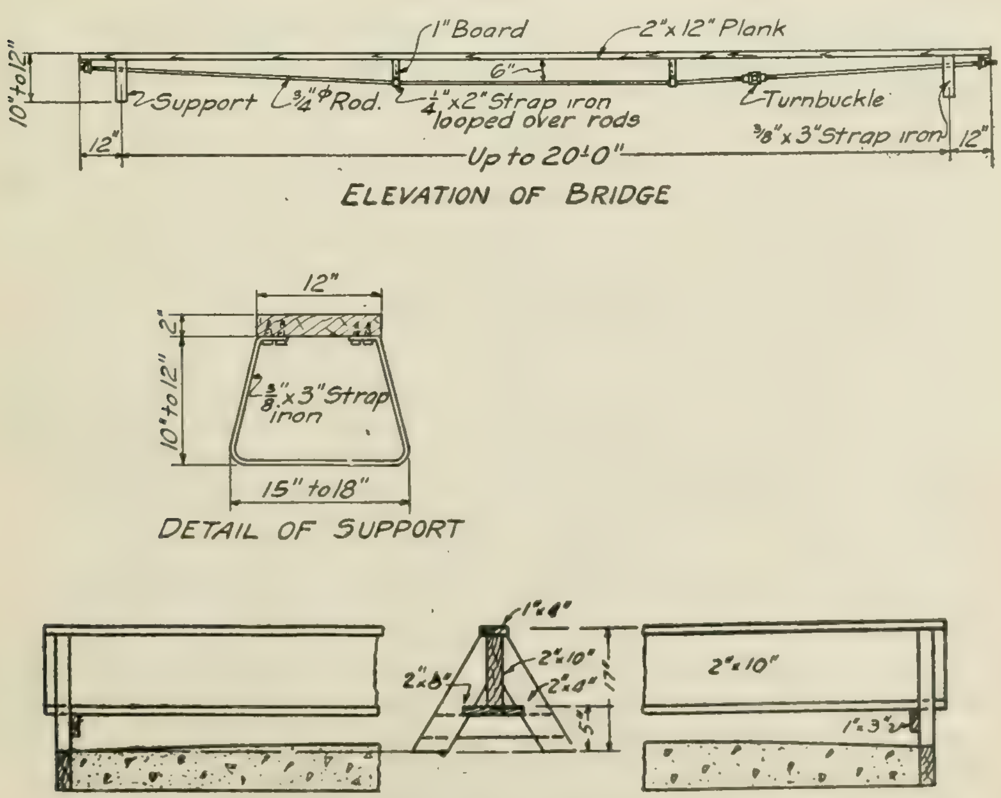


FIG. 25.—ALTERNATE BRIDGES FOR PAVEMENTS UP TO 20 FEET WIDE.

PLACING REINFORCEMENT.

For pavement reinforcement wire mesh or expanded metal with an effective weight of approximately 25 lb. per 100 sq. ft. is recommended.

(*Third Paragraph.* Cut out the first sentence reading)

"The effective areas," etc.

(*Fifth Paragraph.* Cut out in the third sentence)

"For this purpose the American Steel and Wire Co. has" (and substitute the words) Certain companies have.

Section 38.—(After the first paragraph under this section insert the following.)

Reinforcement may consist of bars ¾ in. round or ⅝ in. square bars placed in the following manner: Where joints are made either of the expansion or contraction type, a bar shall be laid transversely across the road parallel with the joints and not less than 6 in. or more than 8 in. from either side of the joint placed within 2 in. of the top of the slab. The ends of the rods at the corner shall be bent at right angles 6 in. from the end. Longitudinal rods shall be placed not less than 6 in. or more than 8 in. from the side of the slabs and laid within 2 in. of the top.

If convenience requires that more than one length of rod shall be used in a given slab, they shall be lapped at least 10 in. Additional transverse rods shall be used as may be indicated on the plans.

If the pavement is constructed without joints, either expansion or contraction, longitudinal rods shall be laid as above described, and transverse rods placed as shown upon the plans.

For pavements 30 ft. or over in width, circumferential reinforcement is to be laid in addition to mesh reinforcement, the rods to be placed as here provided or as may otherwise be shown upon the drawings.

Section 39.—(Cut out the second sentence and substitute the following.)

Machines for finishing concrete surfaces are recommended. The concrete should have a consistency as measured by the cylinder test of not greater than a 2-in. slump. Machines shall be so constructed and operated that they will strike off and thoroughly tamp the concrete, and move over the same area repeatedly, giving a contact and finished surface which shall be true to grade and crown absolutely free from porous places and the surface in general shall have a uniform appearance.

Section 40.—(Cut out the last sentence of the first paragraph, also cut out the second and third paragraphs and substitute the following.)

Concrete shall be brought to a proper contour in the following manner:

On roads and streets not over 20 ft. in width, and where a finishing machine is not used the concrete shall first be tamped, then struck off with a template, shaped to the proper crown; the striking edge of which shall be of steel. The template is to be moved continuously over the surface in a sliding motion. Any inequalities or holes shall be filled immediately with concrete from the last batch deposited.

On pavements wider than 20 ft., concrete may be distributed by a lute

or toothless rake so operated as to place the concrete to the finished section. Where luting is used steel pins shall be driven, with their tops set accurately to elevation to the finished surface; pins to be placed not over 8 ft. apart, measured either transversely or longitudinally of the road and in no instance to have less than three pins across the road, one of which shall be at the center and one each at the quarter points.

Concrete adjoining transverse joints shall be dense in character and shall be finished with a wood float which is divided through the center so as to permit finishing on both sides of the joint at the same time. When sure that the concrete on both sides of the joint is at the same elevation extending to the adjacent surface of the pavement, it shall be tested by means of a wooden straight edge not less than 8 ft. long. It shall be notched in the center and cut so that the joint material will not interfere with the straight edge coming in contact with the concrete. This straight edge, which should be shod with a strip of steel, shall be used as a strike-board or template to secure proper contour of the surface of the concrete adjacent to all joints.

As soon as possible after the concrete has been struck off it shall be rolled with an approved metal roller, having a smooth, even surface, approximately 6 ft. in length, not less than 8 in., nor more than 12 in. in diameter, and weighing approximately 100 lb. (Figs. 31-32.) On pavements less than 20 ft. in width, the roller may be operated with a handle, which shall be at least 2 ft. longer than the width of the pavement, and all rolling shall be done from one side of the slab. On pavements 20 ft. and more in width, the roller shall be provided with two bails, to which ropes shall be attached, and the roller pulled across the pavement. The roller shall be operated in such a manner that it advances along the pavement about 2 ft. for each time across. The roller shall pass from one edge of the pavement to the other, care being taken not to run the roller over the side forms so that earth or other foreign material will adhere to it. After the roller has covered a given area in the manner described, the same area shall be similarly covered by the roller not less than three times at intervals of fifteen to forty minutes, and as many times additional as may be necessary to remove excess water.

After the rolling has been completed the pavement shall be finished by two applications of a belt made of canvas or rubber belting, not less than 6 in. wide and not less than 2 ft. longer than the width of the pavement. The belt shall be applied with a combined cross-wise and longitudinal motion. For the first application vigorous strokes at least 12 in. long shall be used, and the longitudinal movement of the belt along the pavement shall be very slight. The second application of the belt shall be immediately after the water glaze or sheen disappears, and the stroke of the belt shall be not more than 4 in., and the longitudinal movement shall be much greater than for the first belting.

Section 42.—(Cut out the second paragraph and substitute as follows.)

After finishing the concrete, it shall be covered with canvas until it

has hardened sufficiently to permit building earth dams across the pavement to be placed at the joints, where joints are used, and along the edges where no curb is built. The surface of the concrete is then to be covered with water to a minimum depth of 2 in., which shall remain upon the pavement for at least ten days, when the water shall be drawn off and the pavement left exposed for at least four days before it shall be open to traffic, and in cool weather for such additional time as may be determined by the engineer. No traffic shall be permitted to use a pavement until all earth coverings have been completely removed. On portions of the pavement where the grade is such as to make ponding impracticable the surface shall be covered with earth for a depth of not less than 2 in. and kept wet for at least ten days.

Section 44.—(Fig. 31 to be corrected as follows.)

1/8 in. x 26 in. x 72 in. galvanized plate should be made to read 1/8 in. x 35 in. x 72 in. galvanized plate.

COMMITTEE ON CONCRETE ROADS AND PAVEMENTS,
H. ELTINGE BREED, *Chairman.*

REPORT OF COMMITTEE ON SIDEWALKS AND FLOORS.

The committee has been able to hold but one meeting during the year. At this meeting, corrections in the proposed revised specifications for sidewalks and for floors were discussed, also certain additions, among them, the roller method of finishing one-course sidewalks. Since then, on further consideration, the following changes* have been agreed upon and are submitted for the approval of the convention to be sent later to letter ballot of the Institute, together with the proposed revised specifications printed in the Proceedings of the last convention.

The committee wishes to state that plans are nearly complete for the series of wear tests of concrete floor mixtures and methods of finishing, mentioned in last year's report, to be undertaken at the Structural Materials Research Laboratory at Lewis Institute, Chicago. These tests will cover a wide range of mixtures, sizes of aggregates and consistencies, besides those commonly employed in concrete floor construction and the wear tests will be made with the Talbot-Jones rattler which has been used for similar tests covering concrete road construction. The results of this investigation, together with those of service tests now being made at the Bureau of Standards on a number of methods of floor surface treatment, should provide valuable material for a report on the wearing resistance of concrete floor surfaces, the treatment of unsatisfactory floors and methods of securing the best results.

The committee, therefore, requests that it be continued for the ensuing year.

D. A. Abrams,
E. D. Boyer,
P. M. Bruner,
R. S. Greenman,
J. C. Pearson,
W. M. Rynerson,
W. S. Tait,
J. E. Freeman, *Chairman.*

* Changes in specification as adopted by letter ballot September 1, 1919, are printed on p. 413.

REPORT OF COMMITTEE ON BUILDING BLOCK AND CEMENT PRODUCTS.

The Committee on Building Block and Cement Products respectfully submits the following report:

During the past twelve months this committee has held five meetings. Existing specifications on concrete products were considered and no reason was found for recommending any changes in such specifications.

After careful consideration the committee drafted proposed standard specifications for the manufacture of concrete roofing tile as per the copy attached* and we recommend that the specifications be referred to the Institute for adoption. This work is in accordance with the procedure outlined in the last annual report, for this committee.

We suggest that the name of this committee be changed to read "Committee on Concrete Products." We make this recommendation for the sake of brevity and have used the word "concrete" in place of the word "Cement" as this is a "Concrete Institute" and the products are more correctly described by the word "concrete."

This report and proposed specifications have been submitted to the seven members of this committee for letter ballot with the following results:

Voting for the specifications	6
Voting against the specifications	0
Not voting	1
Voting for the report	6
Voting against the report	0
Not voting	1

R. F. Havlik, *Chairman*
W. R. Harris, *Secretary*.

* Specification as adopted by letter ballot, September 1, 1919, printed on p. 415.

CHANGES IN PROPOSED REVISED SPECIFICATIONS FOR CONCRETE SIDEWALKS AND PROPOSED REVISED SPECIFICATIONS FOR CONCRETE FLOORS.*

(The page numbers refer to the Proceedings of the American Concrete Institute, Vol. XIV, 1918.)

Pages 489, 496:

Change the first sentence under "Fine Aggregate" to read:

"Fine aggregate shall consist of natural sand or screenings from hard, tough crushed rock or gravel consisting of quartz grains or other hard material, clean and free from any surface film or coating and graded from fine to coarse, with the coarse particles predominating."

Change third sentence under "Fine Aggregate" to read:

"Fine aggregate shall not contain injurious vegetable or other organic matter as determined by the colorimetric test nor more than five (5) per cent by volume, of clay or loam."

Page 490:

Change first sentence under "Coarse Aggregate" to read:

"Coarse aggregate shall consist of clean durable crushed rock, pebbles or crushed blast furnace slag, graded in size, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. The slag shall not contain more than one and one-half (1½) per cent of sulphur as sulphides."

Page 497:

Change second sentence under "No. 1 Aggregate for Wearing Course" to read:

"It shall pass when dry a screen having three-eighths (⅜) inch openings and not more than ten (10) per cent shall pass a screen having four (4) meshes per linear inch."

Page 495.

Change third sentence under "Placing" to read:

"The forms shall be filled and the concrete brought by means of a strikeboard to a surface one-quarter (¼) inch above the established grade (to allow for compacting by roller in finishing.)

Strike out present paragraph under "Finishing" and substitute the following paragraphs:

"After the concrete has been brought to a surface approximately one-quarter inch (¼ in.) above the established grade, it shall be compacted with a metal roller having a smooth surface and a diameter of from 5 to 12 in. in a manner to displace the surplus water from the surface

* Adopted by letter ballot, Sept. 1, 1919.

of the concrete. The rolling shall be continued at intervals of 15 to 40 min. until all excess water is removed.

"Unless protected by metal the surface edges of all slabs shall be rounded to a radius of one-half (½) inch. (Cross out the following sections except for the surface desired).

"(*a*) Smooth Surface: Following the rolling above described, the surface of the concrete shall be troweled with a steel trowel to a smooth even surface free from depressions or irregularities of any kind. Excessive working of the surface with the trowel shall be avoided.

"(*b*) Medium Rough Surface: Following the rolling above described, the surface shall be floated with a wooden float only, producing an even, gritty finish. On wide sidewalks after rolling, the finishing may be done with two applications of a canvas belt, not less than 6 in. wide, and about 2 ft. longer than the width of the sidewalk. For the first application, the belt shall be drawn across the surface with vigorous strokes at least 12 in. long, and moved ahead very slightly at each stroke. The second application shall be given immediately after the water glaze or sheen disappears, and the stroke of the belt shall be not more than 4 inches, while the longitudinal motion shall be greater than during the first application.

"(*c*) Rough Surface: Following the rolling above described the surface shall be floated with a wooden float leveling all sags and producing an even surface. After being floated in this manner the roughening shall be done by slapping the surface of the concrete with the face of the float, raising the float vertically from the concrete after each stroke."

AMERICAN CONCRETE INSTITUTE.

STANDARD SPECIFICATIONS NO. 22.

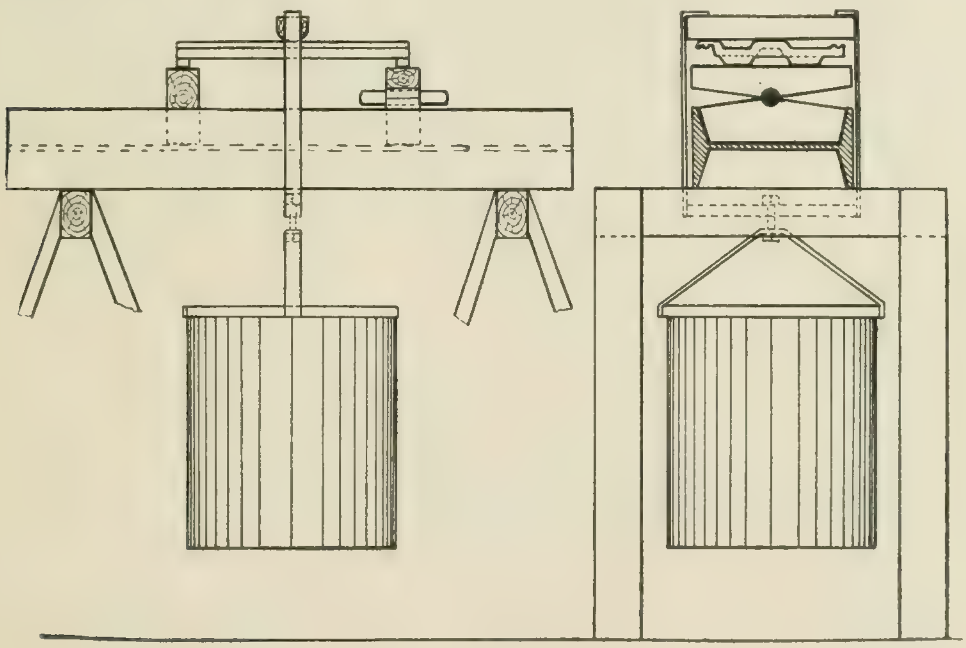
STANDARD SPECIFICATIONS FOR THE MANUFACTURE OF CONCRETE ROOFING TILE.*

1. These specifications apply to concrete roofing tile approximately 9 by 15 in. over all.

2. Concrete roofing tile meeting the requirements of these specifications may be used in building construction.

3. (*a*) Concrete roofing tile must be subjected to load test.

(*b*) Tests shall be made on full size samples. At least 10 samples must be provided for the purpose of testing and must represent the ordinary commercial product.



APPARATUS FOR TESTING CONCRETE ROOFING TILE.

4. *Load.*—The breaking load shall average not less than 150 lb. per tile when the load is applied in accordance with the method described below. Lots of tile intended for building construction may be rejected if more than 10 per cent of the samples tested break at loads of less than 100 lbs.

5. (*a*) *Method of Testing.*—Tile shall be tested with weather face up. The tile shall be supported under the lugs near the ends if the tile have lugs and in no event shall the span be less than 13 in. The support under one end shall be rigid and the support under the other end shall rest on a roller bearing to allow for variation in the under surface of the tile.

(*b*) The load shall be applied in the center of the tile by placing a rigid bar having a semi-circular bearing across the tile midway between the points

* Adopted by letter ballot, Sept. 1, 1919.

of support. From this cross bar shall be suspended a bucket-like receptacle which shall be loaded with shot, sand or other suitable material until the tile breaks. The method of loading is shown in the accompanying diagram.

6. *Cement.*—Portland cement shall be used in the manufacture of concrete roofing tile and shall meet the requirements of the current Standard Specifications for portland cement adopted by the American Society for Testing Materials.

7. *Aggregates.*—Aggregates used in the manufacture of concrete roofing tile shall be of such a nature as will produce the quality of the tile required by these specifications.

ANNUAL REPORT OF THE BOARD OF DIRECTION.

The headquarters of the Institute this year have been continued in Boston with Henry B. Alvord as Secretary.

The Institute has published Vol. XIV of the Proceedings of the Institute, of the Fourteenth Annual Convention, held last June in Atlantic City, also the preprints of the present Convention.

The Treasurer's report, as shown by the Auditor's report, will show that previous to the last Convention there was a balance of $4,378.24, whereas on May 31, 1919, the balance, including $3,000 Liberty Notes, was $5,288.54. Previous to the Convention two years ago the similar balance was $3,922.16.

Despite the continued war conditions and readjustments consequent upon the declaration of the armistice, the membership has shown an increase. It is particularly urgent, however, in this period of readjustment and unsettled conditions that the loyalty of the members of the Institute be maintained in the future as in the past. With this assured, the Institute can take its rightful place in its chosen field.

Respectfully submitted,

THE BOARD OF DIRECTION.

W. K. HATT,

President.

June 28, 1919.

ANNUAL REPORT OF TREASURER.

At the annual convention the Treasurer submitted the report of the auditors, Cooley & Marvin Co., Accountants and Engineers, of Boston, Mass., for the twelve months ending May 31, 1919. This report is as follows:

June 18, 1919.

The American Concrete Institute,

6 Beacon St., Room 428, Boston, Mass.

DEAR SIRS:

In accordance with your instructions we have made an examination of the books and records of the American Concrete Institute for the year ended May 31, 1919, for the purpose of verifying the cash transactions of the period and presenting the financial condition of the American Concrete Institute at that date.

We submit herewith two exhibits, as follows:

Exhibit A. Statement of Condition as at May 31, 1919.

Exhibit B. Statement of Receipts and Disbursements for the year ended May 31, 1919.

The cash in bank, amounting to $2,288.54, as shown by the cash book, was verified by reconciliation with the statement rendered by your deposi-

tary as at May 31, 1919. During our examination paid checks, all of which were properly approved by your President and Secretary or Acting Secretary, were seen for all disbursements.

The remaining items on the statement of condition are shown in accordance with the records, and have not been further verified by us.

We have not attempted to determine the income which should have been derived during the year, and have restricted our examination to accounting for the disposition of all cash shown to have been received.

We hereby certify:

1. That all cash shown to have been received has been accounted for, and that we have seen satisfactory evidence of payment for all disbursements.
2. That the cash in bank, amounting to $2,288.54 at May 31, 1919, was on deposit at that date.
3. That the Statement of Condition (Exhibit A) is in accordance with the records and, subject to the foregoing comments, in our opinion properly presents the financial condition of the American Concrete Institute at May 31, 1919.

Very truly yours,

COOLEY & MARVIN CO.,
Accountants.

Boston, Mass.

Exhibit A.

AMERICAN CONCRETE INSTITUTE.

STATEMENT OF CONDITION.

As at May 31, 1919.

ASSETS.

Cash in bank		$2,288.54
U. S. Victory Notes		3,000.00
Accounts receivable—dues		1,077.00
Accounts receivable—miscellaneous		53.50
Inventories:		
Journals	$291.60	
Proceedings	994.20	
Supplies	46.60	
Total inventories		1,332.40
Total assets		$7,751.44

SURPLUS

Surplus	$7,751.44

Exhibit B.

AMERICAN CONCRETE INSTITUTE.

Statement of Cash Receipts and Disbursements.

As shown by the Cash Book

For the year ended May 31, 1919.

Receipts.

Balance May 31, 1918		$4,378.24
Dues	$4,534.47	
Subscriptions	1,791.70	
Proceedings	830.40	
Bindings	165.21	
Miscellaneous	125.30	
Total receipts		7,447.08
		$11,825.32

Disbursements.

Proceedings	$3,542.33	
Liberty Bonds	3,000.00	
Office expense	1,624.88	
Convention	793.52	
Membership campaign	232.83	
Board meeting expense	218.60	
Office printing	67.68	
Miscellaneous expense	56.94	
Total disbursements		9,536.78
Balance May 31, 1919—Exhibit A		$2,288.54

ABSTRACT OF MINUTES OF MEETINGS OF THE BOARD OF DIRECTION.

MEETING AT THE OFFICE OF TURNER CONSTRUCTION CO., NEW YORK CITY, WED., DEC. 11, 1918, 3 P. M.

Present: President W. K. Hatt, Messrs. Turner, Wason, Humphrey, Wight, and Alvord.

The reading of the minutes of the previous meeting at Atlantic City was dispensed with.

In the absence of the Treasurer, Mr. Alvord presented a financial statement as of Dec. 11, 1918, showing a balance in the bank of $2,734.01, with outstanding bills for Vol. XIV, just issued, amounting to $2,153.20.

The Secretary's report was read and accepted.

F. C. Wight gave an outline report covering the work in publishing Vol. XIV. The volume is now in the hands of the members. It was the consensus of opinion that Vol. XIV is of high technical value and doubtless the most valuable volume of Proceedings which the Institute has as yet produced.

R. L. Humphrey reported on the unpublished portion of Vol. IX. On account of his intimate connection with war work at Washington, he stated that it would not be possible for him to superintend the publishing of the same until the pressure of war work is relieved.

The committee appointed to select the papers to receive the Wason Medal Award, covering the years 1916-17-18, reported through Mr. Wight, Chairman. Their recommendations, which were adopted by the Board, are as follows:

> 1916—"The Influence of Temperature on the Strength of Concrete," by A. B. McDaniel.
>
> 1917—"History and Present Status of the Concrete Pile Industry," by Major C. R. Gow.

The selection for 1918 has not been made as yet, but will be submitted to the Board by letter ballot as soon as received by the Secretary.

The awards are to be made to the authors at the next Convention.

The committee was requested by the donor to compose the inscription which is to appear on the medal.

Voted to hold the next Convention at Atlantic City in June, 1919, in conjunction with the annual meeting of the American Society for Testing Materials, the exact date to be fixed by the Executive Committee. Mr. Humphrey wishes to be recorded against this motion.

The question of permanent headquarters was discussed at some length. Various suggestions were offered, but it was left informally that the Institute should continue on the present basis until a better solution of the problem was found.

MEETING AT THE OFFICE OF TURNER CONSTRUCTION CO., NEW YORK CITY, APRIL 4, 1919, 3 P. M.

Present: President Hatt, Messrs. Turner, Lesley, Boyer, Thompson, Wight, and Alvord.

The reading of the minutes of the previous meeting was dispensed with.

The Secretary's report was read and accepted.

The Treasurer's report, showing a balance of $5,039.59 in the bank, April 4, 1919, was read and accepted.

Voted to authorize the Treasurer to invest $3,000 in United States Bonds.

The Committee on Award of the Wason Medal for the year 1918 reported through its chairman, Mr. Wight, with the recommendation that the medal be awarded to Prof. Duff A. Abrams for his paper on "Effect of Time of Mixing on the Strength and Wear of Concrete." This report was adopted by the Board.

The dates were fixed for the next Convention at Hotel Traymore, Atlantic City, as June 27th and 28th.

Voted that a committee be appointed to prepare a suitable medal to be presented to Mr. Leslie Comyn in recognition of his distinguished service in the construction of the concrete ship "Faith." Messrs. Turner, Boyer and Wight were appointed by the Chair.

Voted to accept the quotation of John C. Winston Co. for publishing Vol. XV. Voted to request Mr. Wight to act as editor of this volume.

Voted to appoint B. S. Pease as representative of the Institute on a Joint Committee with the American Concrete Pipe Association, in the preparation of a set of standard specifications for concrete pipe.

The President was authorized to represent the Institute, with power to act, at a meeting with the National Service Committee of the Engineering Council in Chicago, Illinois, April 23-25, concerning the formation of a Department of Public Works.

The report of the Nominating Committee was received, and, on motion, was referred to the Executive Committee for action.

MEETING AT THE HOTEL TRAYMORE, ATLANTIC CITY, JUNE 28, 1919, 11 A. M.

Present: President Hatt, Messrs. Turner, Thompson, Wason, Anderson, Lesley, Boyer, Kinney, Humphrey, Wight, and Alvord.

The minutes of the preceding meeting of the Board, held April 4, 1919, was read and accepted.

The Treasurer's report was read and accepted. This was later presented to the Convention.

The Secretary's report was read and accepted.

It was reported that the material for the remaining monthly journal, 1915, edited by R. L. Humphrey, is now ready for the printer and will be issued at an early date.

Henry C. Turner was nominated to receive the Leslie Comyn Medal for

Mr. Comyn, who was unable to be present to receive it in person. It was voted to have this medal photographed for reproduction in Vol. XV.

The resignation of Mr. John E. Conzelman as Chairman of the Committee on Industrial Concrete Houses was accepted.

Voted to hold the next Annual Convention in Chicago, during the month of February, 1920, the date to be fixed by the Executive Committee in coöperation with the Concrete Products Association and any other similar associations who may hold their annual meeting in Chicago in February. The selection of headquarters of the Convention was also voted to be left to the Executive Committee with full power.

It was voted to print the membership list and committee list in the forthcoming Volume of Proceedings.

Voted to accept the invitation of the American Society for Testing Materials to coöperate with it and other societies in the formation of specifications for concrete and reinforced concrete, provided the representation of the Institute on the joint conference should be equal to that of each of the other associated societies.

It is also understood that this Committee will have no power to obligate the Institute without the approval of the Board of Direction.

Voted to appoint A. B. Cohen as additional representative with B. S. Pease on the Joint Committee with the American Concrete Pipe Association in the preparation of a set of standard specifications for concrete pipe.

MEETING AT THE HOTEL TRAYMORE, ATLANTIC CITY, JUNE 28, 1919, 6 P. M.

Present: President Hatt, Messrs. Thompson, Turner, Wason, Humphrey, Boyer, Anderson, Kinney, and Alvord.

Voted that the President appoint a committee of three to formulate a plan for permanent headquarters, and report to the Board of Direction at the next meeting.

Voted that the present Executive Committee be continued.

Voted to continue the present Secretary until the next meeting of the Board of Direction.

A vote of thanks was given Mr. Wight for the efficient manner in which he has handled the papers for the present annual meeting.

Voted to form a Committee on Reinforced-Concrete Storage Tanks, appointed by the President.

A Resolution Committee, consisting of Messrs. Humphrey, Kinney and Boyer, was appointed by the President.

LIST OF MEMBERS OF THE AMERICAN CONCRETE INSTITUTE.

** Indicates a Contributing Member.*

*Aberthaw Construction Co., 27 School St., Boston, Mass.
Abrams, Duff A., Lewis Institute, Chicago, Ill.
*Aetna Portland Cement Co., Union Trust Bldg., Detroit, Mich.
Affleck, B. F., 210 S. LaSalle St., Chicago, Ill.
Albright & Mebus, Land Title Bldg., Philadelphia, Pa.
Aldridge, E. V., 210 S. LaSalle St., Chicago, Ill.
*Allentown Portland Cement Co., Allentown, Pa.
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*Alpha Portland Cement Co., Easton, Pa.
Amhursen Construction Co., 61 Broadway, New York City.
American Bureau of Shipping, 66 Beaver St., New York City.
American Can Co., 120 Broadway, New York City.
American Cement Tile Mfg. Co., Oliver Bldg., Pittsburgh, Pa.
American System of Reinforcing, 10 S. LaSalle St., Chicago, Ill.
Archibald & Holmes, Continental Life Bldg., Toronto, Ont.
Asbestos Shingle, Slate and Sheathing Co., Ambler, Pa.
Ash GroveLime and Portland Cement Co., 705 Grand Ave., Temple, Kansas City, Mo.
Ashton, Ernest, Lehigh Portland Cement Co., Allentown, Pa.
Associated Metal Lath Mfgrs., 901 Swetland Bldg., Cleveland, Ohio.
Association of Portland Cement Mfrs., Ltd., Park House, Gravesend, England.
Baker, Hugh J., Majestic Bldg., Indianapolis, Ind.
Ballinger, Walter F., 17th and Arch Sts., Philadelphia, Pa.
Barbour, F. A., 1120 Tremont Bldg., Boston, Mass.
Barrows, Frank G. (Nat. Eng. Corp.), 27 School St., Boston, Mass.
Bartlett, G. S., 208 S. LaSalle St., Chicago, Ill.
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Bell, James C., 12 Twentieth St., Elmhurst, L. I.
Bent Bros., Central Bldg., Los Angeles, Calif.
Bentley, A., & Sons Co., Toledo, Ohio.
Berger, Bernt, 150 Nassau St., New York City.
Berger Mfg. Co., Canton, Ohio.
Best, Byron G., Ironwood, Mich.
Billings, A. W. K., U. S. Naval Aviation Forces, via Postmaster, N. Y.
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Birmingham Slag Co., 1607–16 Jeff. Co. Bk. Bldg., Birmingham, Ala.
*Blaw-Knox Co., Pittsburgh, Pa.
Bodycomb, Walter C., 37 Wall St., New York City.
Bowditch, John, Jr., 105 Broadway, Youngstown, Ohio.
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BROWN, H. WHITTEMORE, University Club, Madison, Wis.
BROWN, PHILIP B., 410 London Bldg., Vancouver, B. C.
BRUFF, JAMES L., Gurney Bldg., Syracuse, N. Y.
BRYANT, HENRY F., 334 Washington St., Brookline, Mass.
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BURNS, HOMER S., Freeport, Texas.
BURROUGHS, H. ROBINS, 24 E. 42d St., New York City.
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BUSH, ADAM L., Emergency Fleet Corp., 140 N. Broad St., Phila., Pa.
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CASE SCHOOL OF APPLIED SCIENCE, Cleveland, Ohio.
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CHAPPELL, FRANK W., 729 Birch Bldg., Dallas, Texas.
CHUBB, JOSEPH H., 836 Security Bank, Minneapolis, Minn.
CLARKE, T. W., 70 Kilby St., Boston, Mass.
CLEMENT, F. H., & CO., Land Title Bldg., Philadelphia, Pa.
*CLINCHFIELD PORTLAND CEMENT CORPORATION, Kingsport, Tenn.
*CLINTON WIRE CLOTH CO., Sears Bldg., Boston, Mass.
COBB, LOUIS R., 37 Wall St., New York City.
COHEN, A. B., D., L. & W. R. R., Hoboken, N. J.
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CONDRON CO., 1433 Monadnock Bldg., Chicago, Ill.
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*CORRUGATED BAR CO., Mutual Life Bldg., Buffalo, N. Y.
COWELL, HENRY, LIME AND CEMENT CO., 2 Market St., San Francisco, Calif.
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ELDRIDGE, H. W., New City, Rockland Co., N. Y.
ELLENDT, JOHN Co., Sibley Block, Rochester, N. Y.
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FOLZ, FRANK W., & Co., Cincinnati, Ohio.
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*FRANKLIN STEEL WORKS, Franklin, Pa.
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FRIEBELE, J. F., care of John W. Ferguson Co., Paterson, N. J.
FROEHLING & ROBERTSON, Richmond, Va.
FROST & CHAMBERLAIN, Slater Bldg., Worcester, Mass.
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*GIANT PORTLAND CEMENT Co., Pennsylvania Bldg., Philadelphia, Pa.

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Lake, Simon, Milford, Conn.
Lambert, Walter E., 644 Prudential Bldg., Buffalo, N. Y.
Landor, Edward J., 634 Renkert Bldg., Canton, Ohio.
Lander, R. S., Johnson City, Tenn.
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*Lehigh Portland Cement Co., Young Bldg., Allentown, Pa.
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Leonard, John B., 528 Rialto Bldg., San Francisco, Calif.
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Ley, Fred T., Inc., 495 Main St., Springfield, Mass.
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Lindau, A. E., Corrugated Bar Co., Buffalo, N. Y.
Lober, John B., Land Title Bldg., Philadelphia, Pa.
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Lockwood, Greene & Co., 60 Federal St., Boston, Mass.
Lord, Arthur R., Straus Bldg., Chicago, Ill.
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Martin, Wisner, 6 Huntington Pl., Norwich, Conn.
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Meyer, Morrison & Co., Inc., 149 Broadway, New York City.
*Michigan Portland Cement Co., Chelsea, Mich.
Michigan University Library, Ann Arbor, Mich.
Miller, Charles R., Co., Inc., 556 Suzette St., Memphis, Tenn.
Mingle, J. G., 425 Citizens Bldg., Cleveland, Ohio.
Mitchell, James, Inc., 76 Montgomery St., Jersey City, N. J.
Monks & Johnson, 78 Devonshire St., Boston, Mass.
Montgomery, Hugh, 24 Castle St., Cape Town, S. Africa.
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Murphy, Gerald G., Midland Ry., York Rd., Belfast, Ire.
National Fireproofing Co., Flatiron Bldg., New York City.
*Nazareth Portland Cement Co., Nazareth, Pa.
*Newaygo Portland Cement Co., Newaygo, Mich.
New England Concrete Const. Co., 201 Devonshire St., Boston, Mass.
New Jersey Zinc Co., 55 Wall St., New York City.
Newman, Rolf R., Engineer School, Camp Humphreys, Va.
Northwestern Expanded Metal Co., 37 W. Van Buren St., Chicago, Ill.
Northwestern States Pt. Ct. Co., Mason City, Iowa.
Oakey, Charles W., Eng. Dept. C., R. I. & P. Ry., Chicago, Ill.
Oesterblom, Isaak, American Trading Co., Shanghai, China.
Ogden Portland Cement Co., Eccles Bldg., Ogden, Utah.

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Ord, William, Lakewood Eng. Co. Cleveland O.
Orr, John B., 11th St., Miami, Fla.
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*Ottawa Silica Co., Ottawa, Ill.
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*Penn-Allen Portland Cement Co., Widener Bldg., Allentown, Pa.
*Pennsylvania Cement Co., 30 E. 42d St., New York City.
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Prack & Perrine, 208 Traders Bank Bldg., Toronto, Can.
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Randall, Frank A., 19 S. La Salle St., Chicago, Ill.
*Raymond Concrete Pile Co., 140 Cedar St., New York City.
Reynvaan, A. J., care of United Verde Copper Co., Clarkedale, Ariz.
Rheinstein & Haas, Inc., 21 E. 40th St., New York City.
Richart, Frank E., University of Illinois, Urbana, Ill.
Richmond, Knight C., 10 Weybossett St., Providence, R. I.
Ritter, Louis E., 1707 Marquette Bldg., Chicago, Ill.
*Riverside Portland Cement Co., 640 Title Insurance Bldg., Los Angeles, Calif.
Robinson, C. C., 510 Times-Despatch Bldg., Richmond, Va.
Rogers, Floyd, Newton, Iowa.
Rogers, J. S., Co., Moorestown, N. J.
Rogow, Sydney, 14 Morningside Ave., New York City.
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Saurbrey, Alexis, 414 Wyoming Ave., Scranton, Pa.
Schilling, A., Eastern Potash Corp. Equitable Bldg., New York City.

SCHOFIELD, R. W., Whakatane, New Zealand.
SCHWAN, GEORGE H., Arch 1310 Peoples Bank Bldg., Pittsburgh, Pa.
SECURITY CEMENT AND LIME CO., Hagerstown, Md.
SEWELL, JOHN S., Gantt's Quarry, Ala.
SHELDON, F. P., & SON, Hospital Trust Bldg., Providence, R. I.
SIMPSON, GEORGE, 51 Wall St., New York City.
SIMPSON BROS. CORP., 166 Devonshire St., Boston, Mass.
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SMITH, BLAINE S., 210 S. La Salle St., Chicago, Ill.
SMULSKI, EDWARD, 13 Park Row, New York City.
SNODGRASS, R. D., Truscon Steel Co., New York City.
SOUTHWESTERN PORTLAND CEMENT CO., Herald Bldg., El Paso, Texas.
SPACKMAN, HENRY S., ENGR. CO., 2024 Arch St., Philadelphia, Pa.
SPAULDING, RALPH EDGAR, Suffield, Conn.
SPIKER, WILLIAM C., Candler Bldg., Atlanta, Ga.
SPINOSA, VALL, Const. Exp. Metal Co., Braddock, Pa.
STANDARD OIL OF NEW JERSEY, P. O. Box 16, Elizabeth, N. J.
STEELE & SONS CO., 1600 Arch St., Philadelphia, Pa.
STEVENS, A. W., 244 Madison Ave., New York City.
STEWART, JAMES, & CO., INC., 30 Church St., New York City.
STRUCKMANN, HOLGER, 347 Madison Ave., New York City.
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SWAIN, GEORGE F., Mass. Inst. of Tech., Cambridge, Mass.
TAIT W. STUART, Hydraulic Steelcraft Co., Cleveland, Ohio.
TALBOT, ARTHUR N., 1013 W. California Ave., Urbana, Ill.
TALBOT, KENNETH H., Koehring Machine Co., Milwaukee, Wis.
TANNER, B. M., Idaho Falls, Idaho.
TAYLOR ENGR. CO., Vancouver, B. C.
TEXAS PORTLAND CEMENT CO., Dallas, Texas.
THOMPSON, SANFORD E., 136 Federal St., Boston, Mass.
THOMPSON & BINGER, 280 Madison Ave., New York City.
TOCH BROS., 320 Fifth Ave., New York City.
TORCRETE SHIPBUILDING CORP., 120 Broadway, New York City
TRINITY PORTLAND CEMENT CO., 910 Commonwealth Bldg., Dallas, Texas.
*TRUSCON STEEL CO., Trussed Concrete Bldg., Detroit, Mich.
TUBESING, WM. F., 994 Forty-first St., Milwaukee, Wis.
TUCKER, E. A., CO., 683 Atlantic Ave., Boston, Mass.
TUFTS, ARTHUR, Candler Annex, Atlanta, Ga.
TURNER, C. A. P., 627 First Ave., N., Minneapolis, Minn.
TURNER, JOHN J., & SONS, 65 E. Main St., Amsterdam, N. Y.
*TURNER CONSTRUCTION CO., 244 Madison Ave., New York City.
ULRICH, WM. L., 101 Ridgefield St., Hartford, Conn.
UNDERWRITERS' LABORATORIES, 207 E. Ohio St., Chicago, Ill.
UNITED FIREPROOFING CO., 8 W. 40th St., New York City.

United States Portland Cement Co., 305 Ideal Bldg., Denver, Colo.
*Universal Portland Cement Co., 210 S. La Salle St., Chicago, Ill.
Upham, Charles M., Dover, Del.
Upson, Maxwell M., 140 Cedar St., New York City.
Vandever, Jacob B., 1270 Broadway, New York City.
Van Doorn, J. C., 836 Security Bank Bldg., Minneapolis, Minn.
Van Dyke, C. W., 30 Broad St., New York City.
Voshell, James T., Post Office Building, S. Chicago, Ill.
*Voss, Walter C., Wentworth Inst., Boston, Mass.
Vulcanite Portland Cement Co., 8 W. 40th St., New York City.
Wackwitz, Clarence A., Irvington on Hudson, New York.
Wait, B. H., 347 Madison Ave., New York City.
Waldo Bros., Inc., 45 Batterymarch St., Boston, Mass.
Wallace, W. K., Midland Ry., York Rd., Belfast, Ire.
Warner, Chas., Co., Finance Bldg., Philadelphia, Pa.
Warren, George E., 210 S. La Salle St., Chicago, Ill.
Waterloo Construction Machinery Corp., Waterloo, Iowa.
Watson Engineering Co., 1101 Hippodrome Bldg., Cleveland, Ohio.
Webber Construction Co., care of Fred C. Webber, 61 E. 86th St., New York City.
Weber, Carl, 914 S. Michigan Ave., Chicago, Ill.
Weis, Frank J., Lindsey Bldg., Dayton, Ohio.
Westinghouse Church Kerr & Co., 37 Wall St., New York City.
Whipple, A. D., 111 W. Washington St., Chicago, Ill.
White, F. R., 1430 K St., N. W., Washington, D. C.
White Const. Co., Inc., 95 Madison Ave., New York City.
*Whitehall Cement Mfg. Co., 1736 Land Title Bldg., Philadelphia, Pa.
Wickersham, John H., 53 N. Duke St., Lancaster, Pa.
Widmer, A. D., Century Bldg., St. Louis, Mo.
Wight, Frank C., 10th Ave. at 36th St., New York City.
Wigton, C. B., 552 N. 23d St., New York City.
Williams, David E., 1900 Euclid Ave., Cleveland, Ohio.
Wilson, Herbert C., 518 Main St., Worcester, Mass.
Wilson, Percy H., Land Title Bldg., Philadelphia, Pa.
Wilson's Portland Cement Co., Ltd., Box 269, G. P. O., Auckland, N. Z.
Wisconsin University Library, Madison, Wis.
Wolf, Albert M., 902 Eighteenth Ave., Melrose Park, Ill.
Wolverine Portland Cement Co., Coldwater, Mich.
Woolson, Ira H., 76 William St., New York City.
Worcester, J. R., 79 Milk St., Boston, Mass.
Worcester, R. J. H., Concord, Mass.
Wright & Kowalski, Grove and 19th Sts., Jersey City, N. J.
Yeager & Sons, Danville, Ill.
Young, R. B., 8 Strachan Ave., Toronto, Ont.
Young, R. W., Paullina, Iowa.
Young, Thos. F., Midland Ry., Belfast, Ire.
Zeidler, John L., 401 Francis St., St. Joseph, Mo.

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